Coastal and Hydraulics Laboratory

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Engineer Research and Development Center

Upper Guadalupe River Flood Control Weir, San Jose, California

Billy D. Fuller

November 2003

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Final report

Prepared for U.S. Army Engineer District, San Francisco
San Francisco, CA 94105-2197

ABSTRACT: Tests were conducted on a 1:36 scale model of a portion of the Guadalupe River. The study was designed to investigate the design of a control weir located at the upstream end of a bypass channel. The bypass channel was designed to pass excess flow and prevent flooding for river flows up to a 100-year event. The original weir design was modified to achieve desired flow distribution and control.

Contents

Contents

List of Figures

List of Photos

List of Tables

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
cubic feet	0.02831685	cubic meters
degrees (angle)	0.01745329	radians

Preface

The model study reported herein was authorized by the Office, Chief of Engineers, U.S. Army, at the request of the U.S. Army Engineer District, San Francisco. Points of contact from the San Francisco District were Messrs. Carlos R. Hernandez, William R. Firth, and David V. Doak.

The study was conducted during the period July 2000 to March 2002 at the Coastal and Hydraulics Laboratory (CHL) of the U.S. Army Engineer Research and Development Center (ERDC). The study was conducted under the administrative supervision of Mr. Thomas W. Richardson, Director, and Dr. William D. Martin, Deputy Director, respectively, CHL, and under the general supervision of Mr. Charles H. Tate, Acting Chief of the Inland Hydraulic Structures Branch, CHL. The Principal Investigator for the model study was Mr. Billy D. Fuller, assisted by Messrs. Douglas White, Kevin Pigg, and Bill Katzenmeyer, all of CHL. The model was constructed by personnel of the Department of Public Works under the direction of Mr. Cecil Dillon. Messrs. Fuller, White, and Katzenmeyer prepared this report.

COL James R. Rowan, EN, was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

1 Introduction

Prototype

The project location is on the Guadalupe River in the vicinity of the Willow Glen community in San Jose, CA. The reach being studied is 1,600 ft[1] long and extends from 7,100 ft upstream of the Southern Pacific Railroad crossing to about 700 ft upstream of the Willow Glen Way bridge. The proposed flood-control project consists of the natural channel, a diversion (bypass) channel, for flood flow and a control structure (weir). The project is proposed and designed by the Santa Clara Valley Water District. A vicinity map is included as Figure 1.

Purpose of Model Study

The purpose of the model study was to investigate the original weir design and to document the water-surface elevations along the natural and bypass channels for several river flows. The San Francisco District provided the weir design, along with its location and orientation. The intent of the weir design was to restrict flows up to 1,500 cfs to the natural channel and to provide a flow distribution of 5,000 cfs in the natural channel and 9,600 cfs in the bypass channel during the 14,600-cfs, 100-year event. If the original weir design failed to produce desired results, modification details would be provided to the San Francisco District for review prior to its implementation in the model.

Additional investigations were to measure channel velocities in the vicinity of the weir and the Willow Glen Way bridge, and to identify areas that were likely to experience sediment deposition. These two investigations would provide information for bank and channel protection and would indicate areas that could require significant maintenance.

[1] Units of measurement in this report are shown in non-SI units. A table of factors for converting non-SI to SI units of measurement is presented on page vii.

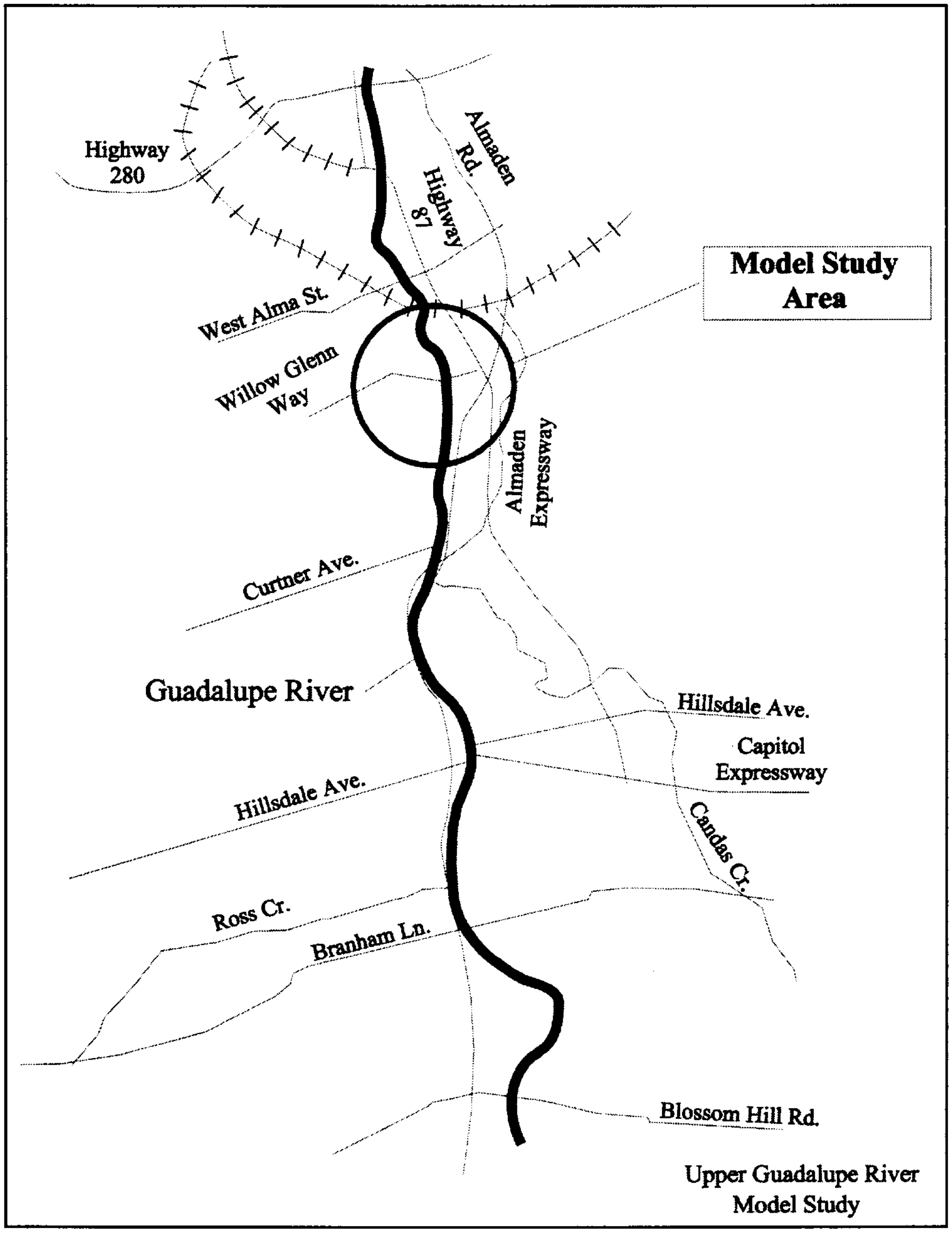

Figure 1. Vicinity map

2 Model

Description

The 1:30 scale model reproduced the weir and the natural and bypass channels beginning at sta 802+00 and extending to sta 786+00 at the downstream extent. The model scale and limits were determined to provide an accurate representation of the flow characteristics necessary to investigate the weir performance. Figure 2 shows the model limits and stationing used in reporting data.

In this model, flow velocities were measured with a pitot tube and discharges were measured with a sharp-crested weir. The limitation of the pitot tube (at this scale) was that velocities of a magnitude less than 2 fps could not be measured. This was not considered a problem in this study because the velocities at which the channel needed protection, as indicated by San Francisco District personnel, were on the order of 20+ fps.

The model discharge was set by measuring the head over a sharp-crested weir (Figure 3). The head versus discharge was calculated using the method described in King and Brater (1976). This approach was questioned during a mid-study review meeting. As a result, a member of the independent technical review team reviewed the discharge calculations. The discharges calculated from the sharp-crested weirs (one for the natural channel and one for the bypass channel) were determined to be within 5 percent when using calculation procedures from the *Water measurement manual* (U.S. Department of the Interior 1984), Rouse Engineering Hydraulics (Rouse 1946), and *The Handbook of hydraulics* (King and Brater 1976) (these calculations are provided in Appendix A). These discharge measurements were within a 10-percent margin of error when scaled to prototype quantities.

Water-surface elevations were measured at the center of the channel on 100-ft intervals. These measurements were made using piezometer taps in the model and point gages over a stilling well for each location. The stilling well dampened the high-frequency changes in water-surface elevation. These measurements were considered average water-surface elevations in subcritical areas of the model. The measurements were average, center of channel water-surface elevations for areas that were supercritical (near the Willow Glen Way bridge and near the control weir at the upstream end of the bypass channel).

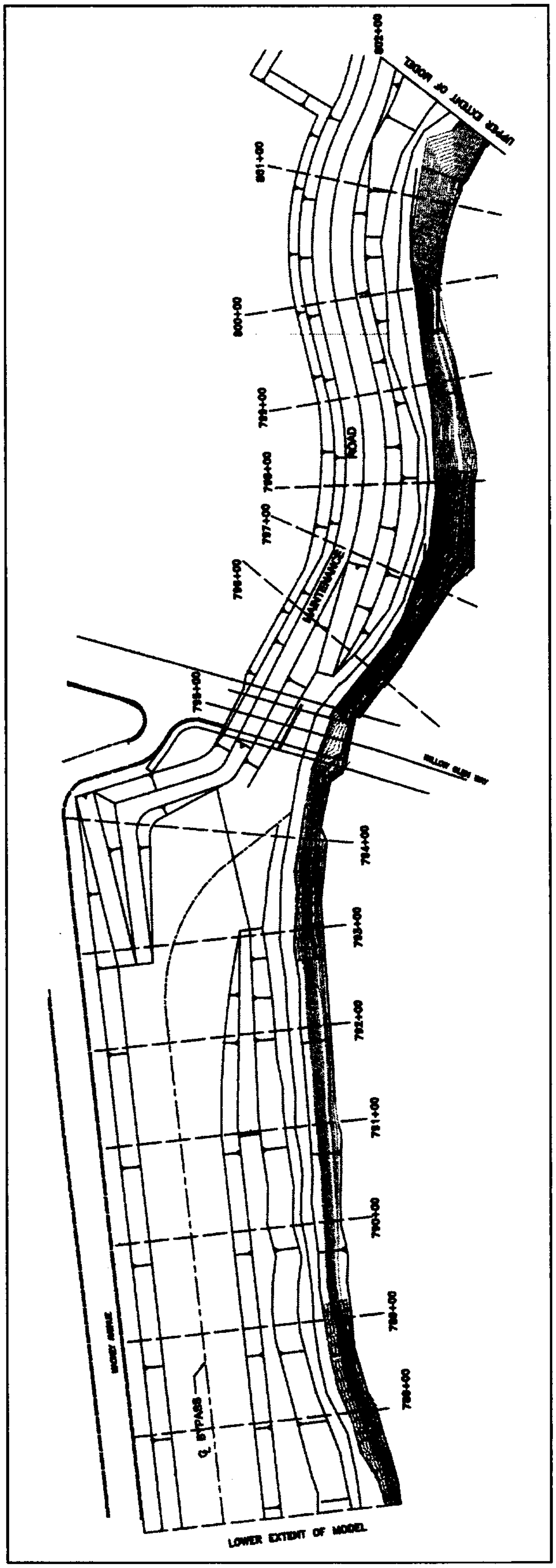

Figure 2. Upper Guadalupe model layout

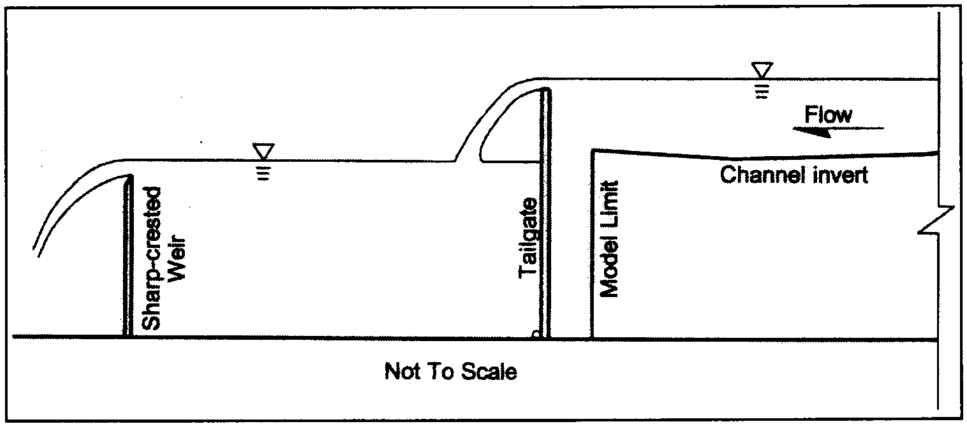

Figure 3. Schematic of tailgate and weir

Model Adjustments

To insure the model provided realistic information, the investigation was set up to document flow conditions for two channel roughnesses: a design roughness and a new channel roughness. These two roughnesses would represent a channel condition some years in the future and a channel condition shortly after construction, respectively. The design roughness was considered to provide conservative water-surface elevations and flow distributions and was therefore used to evaluate the performance and necessary design changes for the weir. The new channel roughness was investigated to demonstrate the sensitivity of the weir to channel roughness and to document conservative velocities for use in designing channel protection.

The San Francisco District provided water-surface profiles representing the two channel roughnesses. They were calculated using the HECRAS numerical code. These water-surface profiles were considered to be boundary conditions and the true prototype response would lie in between these conditions.

Table 1 shows the Manning's n value represented by these boundary roughnesses.

Table 1
Channel Manning's n Values

	Natural Channel	Bypass Channel
Design Roughness	0.050	0.033
New Channel Roughness	0.040	0.028

The model roughness was initially set for the design roughness (n = 0.050 in the natural channel and n = 0.033 in the bypass channel). To achieve the desired roughness, the model discharge was set to 1,500 cfs (lowest event in the study) and the tailwater (from the San Francisco District HECRAS study corresponding to the design roughness and 1,500 cfs discharge) was set (Figure 3). Starting at the downstream end of the channels, roughness (expanded metal) was added to the wetted portion of the channel. The type of the expanded metal was changed until the water surface matched the San Francisco District water surface. Once the 1,500-cfs event roughness was achieved, the model discharge was set to 14,600 cfs (100-year event) and the tailwater (from the San Francisco District HECRAS study corresponding to the design roughness and 14,600 cfs discharge) was set. With the higher flow, the upper portion of the wetted channel required roughness adjustment. Wire mesh was used to provide enough roughness to the channel to match the water surfaces. The new channel roughness was set in the same manner.

Similitude

The accepted equations of hydraulic similitude, based on the Froudian criteria, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and the prototype. A model scale was selected that would provide a model Reynolds number high enough to overcome viscous forces in the model. The general relations expressed in terms of the model's scale or length ratio, L_r, are expressed in Table 2.

Table 2
Scale Relations

Dimension	Ratio	Scale Relation
Length	L_r	1:30
Area	$A_r = L_r^2$	1:900
Velocity	$V_r = L_r^{1/2}$	1:5.477
Discharge	$Q_r = L_r^{5/2}$	1:4929
Time	$T_r = L_r^{1/2}$	1:5.477
Force	$F_r = L_r^3$	1:27,000
Frequency	$f_r = 1/L_r^{1/2}$	1:0.183

Measurements of each of the dimensions or variables can be transferred qualitatively from model to prototype equivalents by means of the scale relations in Table 2. All model data are presented in terms of prototype equivalents.

3 Experiments

Design Roughness

Flow distribution

The weir had to produce two design distributions. Up to 1,500 cfs of all the flow was to remain in the natural channel with no bypass flow. For the 100-year event, 5,000 cfs was to flow in the natural channel and 9,600 cfs in the bypass channel. To meet the first requirement of 1,500 cfs in the natural channel prior to utilizing the bypass, the original weir (Figure 4; Photos 1 and 2) had to be modified by raising the top elevation by 1.25 ft. The second requirement was achieved by reducing the weir length to 128 ft (Figure 5). An alternative weir representing these dimensions and incorporating more aesthetic features was installed in the model (Figure 6; Photos 3-5). This weir was constructed of acrylic plastic and became the final recommended weir design based upon model test results.

After the weir dimensions had been modified to provide desirable flow distributions at the two target discharges (1.2-year event and the 100-year event), flow distributions were measured for intermediate events as shown in Table 3.

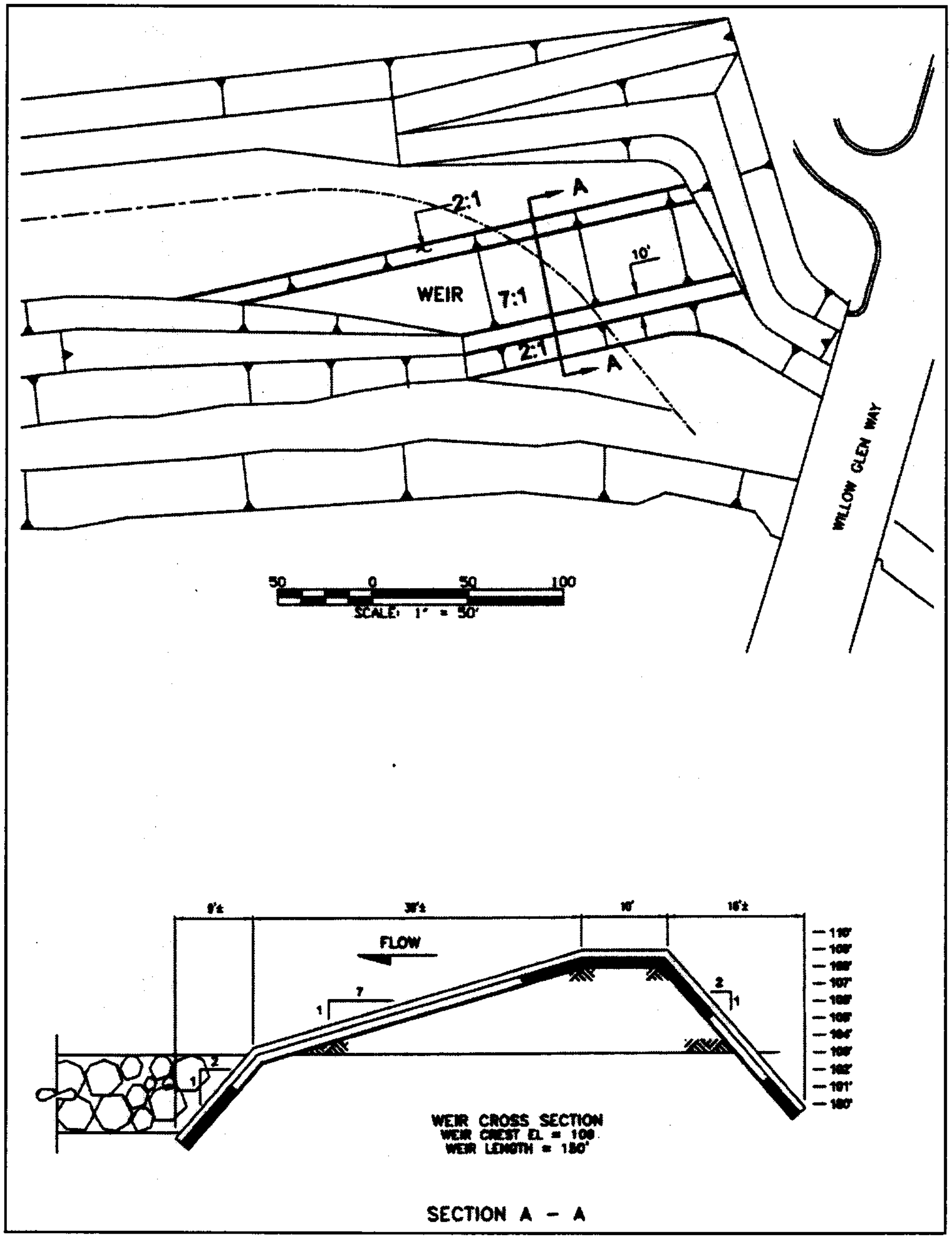

Figure 4. Original weir design

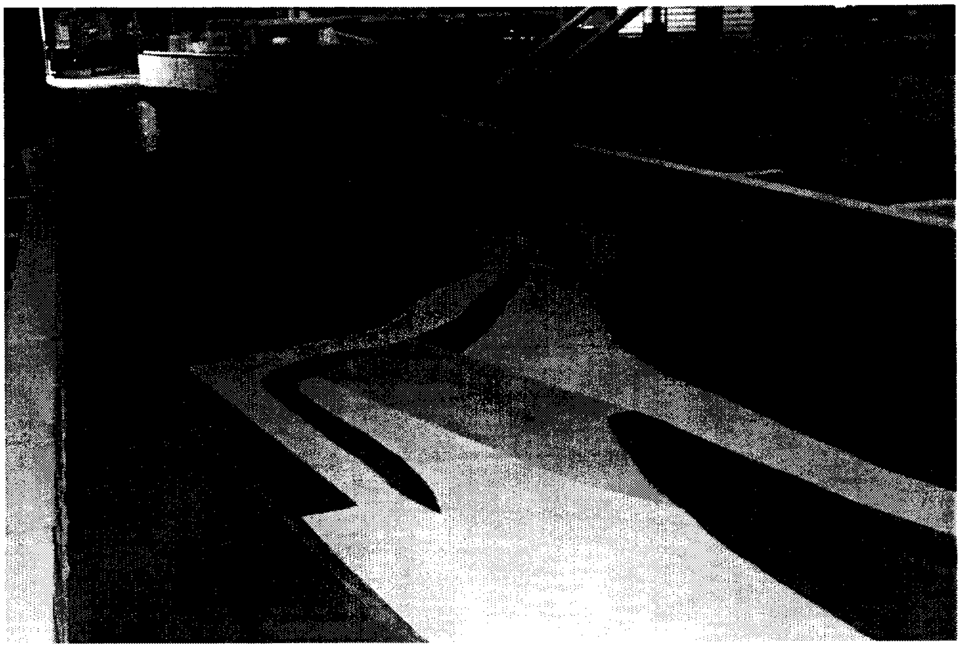

Photo 1. Original weir looking upstream

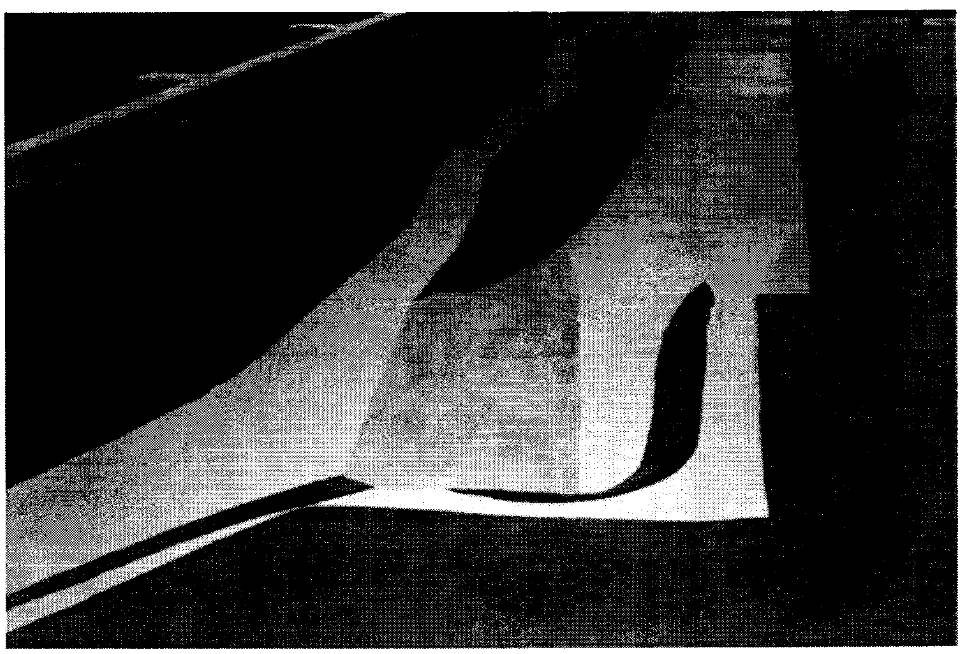

Photo 2. Original weir looking downstream

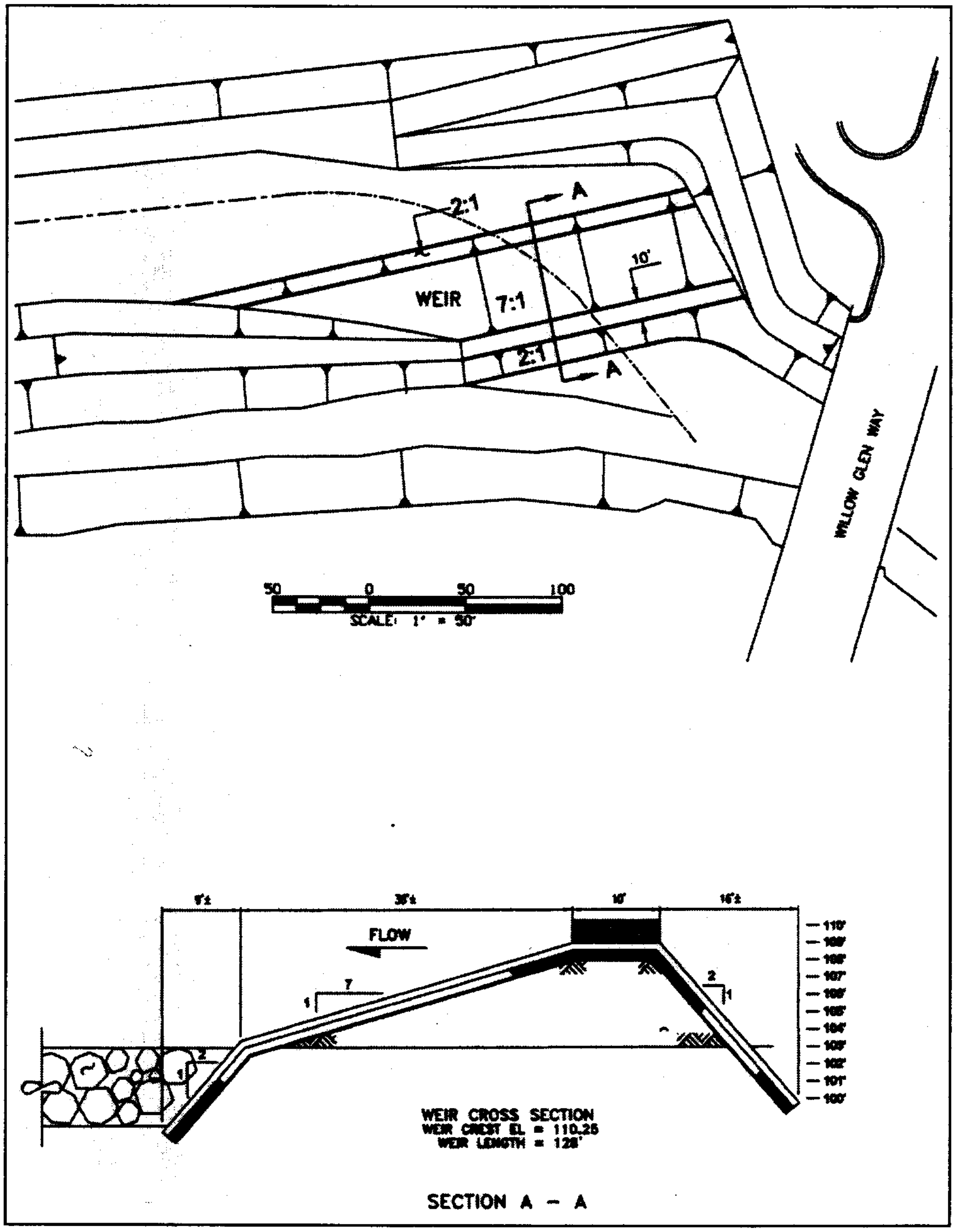

Figure 5. Modified weir design

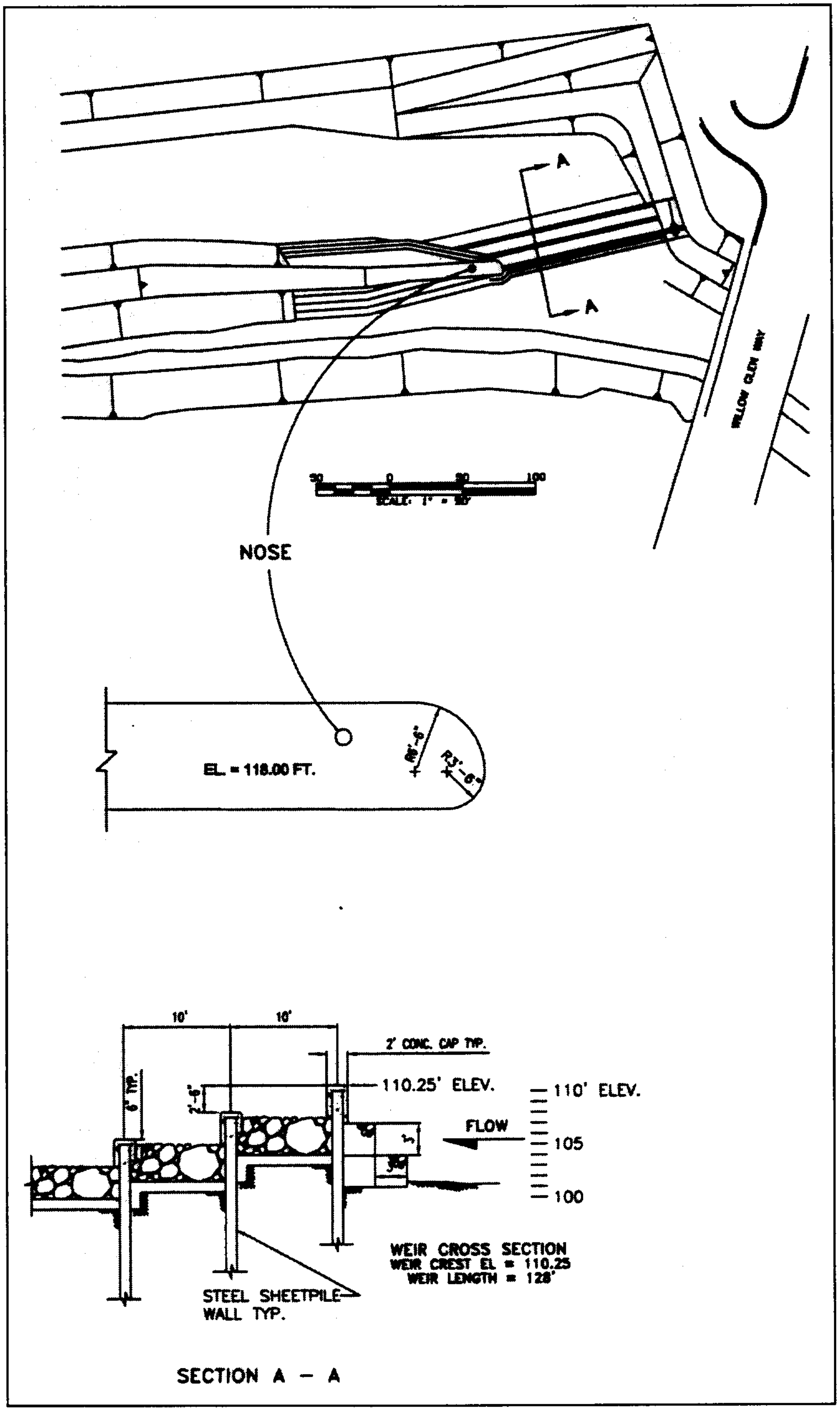

Figure 6. Alternative “final” weir design

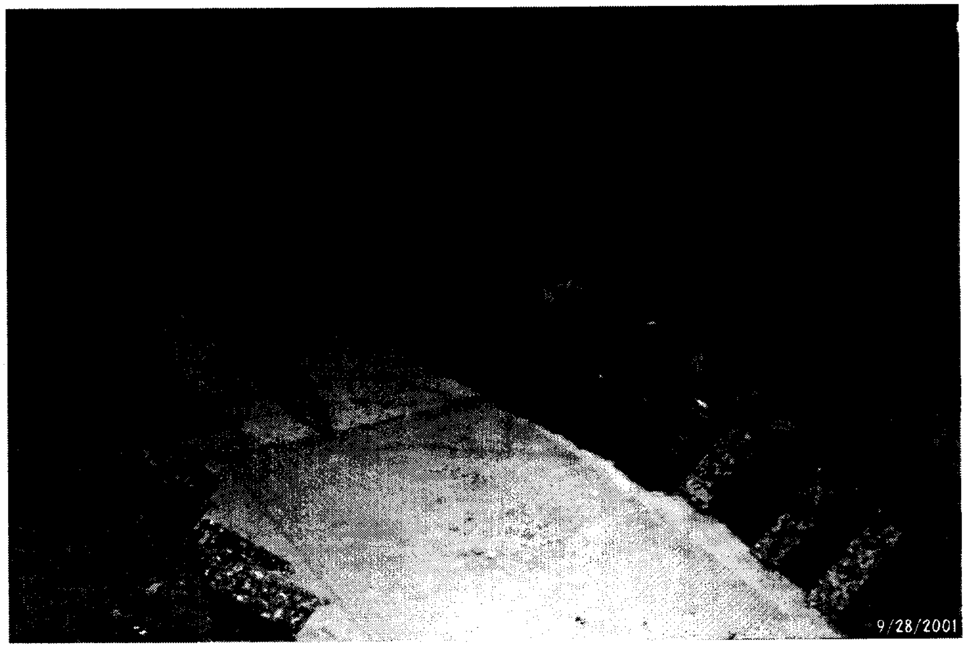

Photo 3. Final weir looking upstream

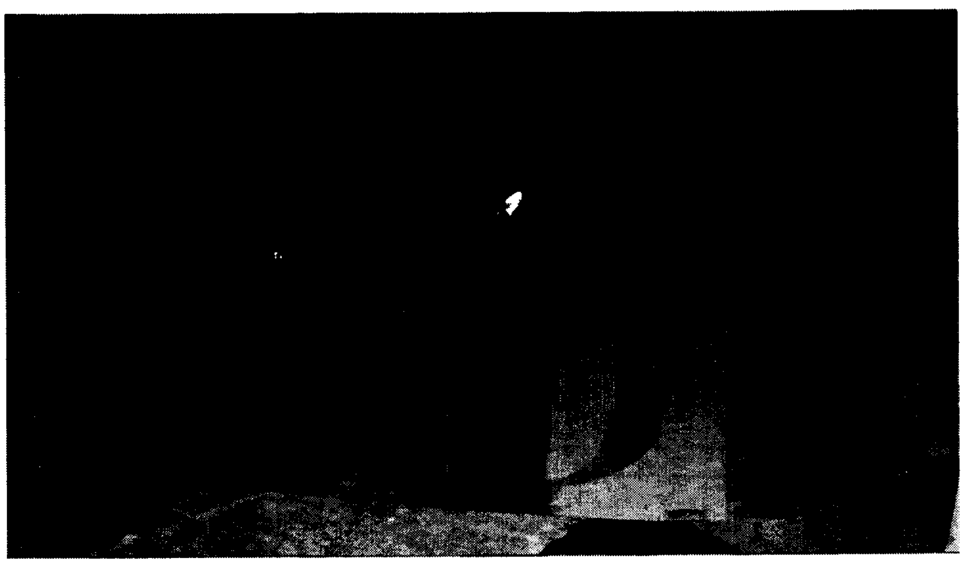

Photo 4. Final weir looking downstream from right bank

Photo 5. Final weir looking downstream from left bank

Table 3 Flow Distributions for Design Roughness			
Event	**Discharge, cfs**	**Natural Channel Discharge, cfs**	**Bypass Channel Discharge, cfs**
1.2-year	1,500	1,500	0
10-year	7,100	3,350	3,750
20-year	9,000	3,800	5,200
100-year	14,600	5,000	9,600

Tailwater sensitivity

The sensitivity of the flow distributions to tailwater elevation was determined for the discharge and tailwater elevation adjustment values shown in Table 4. The tailwater elevation in each channel (natural and bypass) was increased or decreased simultaneously by the amount indicated in the table. The table also indicates the flow distribution changes for the associated change in tailwater elevation. With the exception of the 1,500-cfs flow, higher tailwater elevations caused more flow to enter the bypass channel.

Table 4
Resulting Flow Distribution for Design Roughness

Event	Discharge, cfs	Natural Channel Discharge, cfs	Bypass Channel Discharge, cfs	Tailwater
1.2-year	1,500	1,500	0	-1.0-ft
		1,500	0	**Design**
		1,500	0	+1.0-ft
		1,300	200	+2.0-ft
10-year	7,100	3,450	3,650	-1.0-ft
		3,350	3,750	**Design**
		3,250	3,850	+1.0-ft
		2,750	4,350	+2.0-ft
20-year	9,000	3,900	5,100	-1.0-ft
		3,800	5,200	**Design**
		3,450	5,550	+1.0-ft
		2,950	6,050	+2.0-ft
100-year	14,600	5,000	9,600	-1.0-ft
		5,000	9,600	**Design**
		5,000	9,600	+1.0-ft
		4,600	10,000	+2.0-ft

Water-surface profiles

Water-surface profiles were documented for the four events with four tailwater elevations. These data are presented in Table 5 and drawings in Appendix B. These values are based on point measurements in the center of the channel. In subcritical flow areas of the model, they are considered to be average water-surface elevations.

Table 5
Water-Surface Elevations for Design Roughness

Station	Natural Channel Water-Surface Elevation, NGVD				Bypass Channel Water-surface Elevation, NGVD			
	Event				Event			
	1.2-year	10-year	20-year	100-year	1.2-year	10-year	20-year	100-year
801+00	113.16	117.90	119.28	122.10	No Flow			
800+00	113.19	118.14	119.43	122.07				
799+00	113.04	117.96	119.16	121.65				
798+00	112.68	117.57	118.89	121.56				
797+00	111.90	116.49	117.78	120.36				
796+00	110.51	112.47	113.07	115.17				
795+00	110.43	112.47	112.86	114.90				
794+00	110.34	113.28	113.58	115.44				
793+00	110.16	113.19	113.94	115.80		109.50	111.12	114.99
792+00	109.59	112.86	113.88	116.28		109.53	111.18	115.05
791+00	109.08	112.05	113.22	115.62		109.26	112.65	114.63
790+00	109.14	111.99	113.10	115.59		109.02	110.52	114.36
789+00	108.84	111.51	112.68	115.20		108.96	110.49	114.18
788+00	108.55	111.24	112.50	115.17		109.05	110.58	114.39

Sediment impact

Sediment was introduced in the model to illustrate areas of likely deposition. The magnitude (quantity) of this deposition is not relative to prototype expectations. The procedure used to introduce the sediment was to set the discharge to the 100-year event (14,600 cfs) and deposit 5,000 cu yd of material in 2 hr, let the river flow for 30 min, deposit another 5,000 cu yd at the same rate and then let the river flow for another 30 min. The flow was shut off and the sediment deposits were dusted with cement to allow investigation of sediment effects with several flow conditions. The procedure (as described) and material (sand that simulates about 0.25 in. gravel in the prototype) was the same as that used in Hite (1998). Figure 7 is a plan view of the model showing areas of sediment deposition.

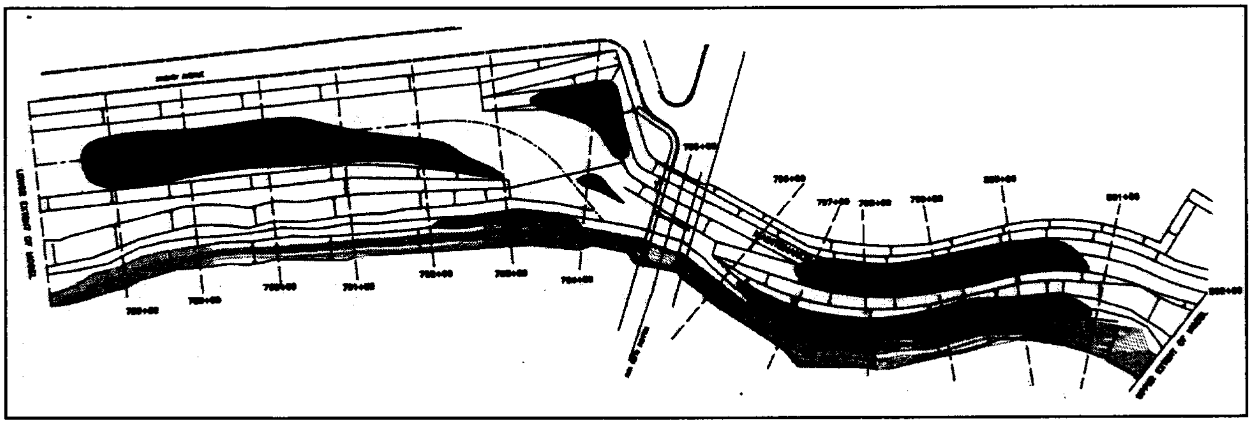

Figure 7. Sediment deposition design roughness

The flow distributions for the design roughness with and without sediment deposition are indicated in Table 6.

Table 6
Flow Distribution for Design Roughness with and without Sediment

Event	Total Discharge, cfs	Discharge Without Sediment Deposits		Discharge With Sediment Deposits	
		Natural Channel	Bypass Channel	Natural Channel	Bypass Channel
1.2-year	1,500	1,500	0	1,200	300
10-year	7,100	3,350	3,750	3,200	3,900
20-year	9,000	3,800	5,200	3,750	5,250
100-year	14,600	5,000	9,600	5,800	8,800

The water-surface elevations for the design roughness with sediment deposition are indicated in Table 7.

Table 7
Water-Surface Elevations for Design Roughness with Sediment Deposition

Station	Natural Channel Water-Surface Elevation, NGVD				Bypass Channel Water-Surface Elevation, NGVD			
	Event				Event			
	1.2-year	10-year	20-year	100-year	1.2-year	10-year	20-year	100-year
801+00	115.29	119.25	120.12	123.09	No Flow			
800+00	115.32	118.74	119.67	122.07				
799+00	115.20	118.35	119.10	121.26				
798+00	113.55	116.64	118.32	121.05				
797+00	111.69	115.47	117.16	120.63				
796+00	111.45	113.07	113.58	115.44				
795+00	111.06	112.92	113.28	114.87				
794+00	110.94	113.37	113.97	115.65				
793+00	110.91	113.94	114.66	116.13		111.60	112.68	114.45
792+00	109.77	112.41	113.94	116.22		110.04	110.91	114.51
791+00	109.56	112.74	113.79	115.85		108.48	110.55	113.94
790+00	108.48	111.09	113.13	115.77		108.57	110.46	113.88
789+00	108.72	111.72	112.89	115.05		109.02	110.43	114.30
788+00	108.57	111.24	112.50	115.17		109.02	110.58	114.45

The differences in water-surface elevation for the design roughness with and without sediment deposition are indicated in Table 8.

Table 8
Water-Surface Elevations Differences for Design Roughness with Sediment Added

Station	Natural Channel, ft				Bypass Channel, ft			
	Event				Event			
	1.2-year	10-year	20-year	100-year	1.2-year	10-year	20-year	100-year
801+00	2.13	1.35	0.84	0.99	No Flow			
800+00	2.13	0.60	0.24	0.00				
799+00	2.16	0.39	-0.06	-0.39				
798+00	0.87	-0.93	-0.57	-0.51				
797+00	-0.21	-1.02	-0.62	0.27				
796+00	0.94	0.60	0.51	0.27				
795+00	0.63	0.45	0.42	-0.03				
794+00	0.60	0.09	0.39	0.21				
793+00	0.75	0.75	0.72	0.33		2.10	1.56	-0.54
792+00	0.18	-0.45	0.06	-0.06		0.51	-0.27	-0.54
791+00	0.48	0.69	0.57	0.23		-0.78	-2.1	-0.69
790+00	-0.66	-0.90	0.03	0.18		-0.45	-0.06	-0.48
789+00	-0.12	0.21	0.21	-0.15		0.06	-0.06	0.12
788+00	0.02	0.00	0.00	0.00		-0.03	0.00	0.06

New Channel Roughness

Flow distribution

The flow distributions for the new channel roughness were recorded to document the distributions associated with a lower water surface produced by the new channel Manning coefficient (Table 1). The resulting distributions are shown in Table 9. Less flow was passed to the bypass channel for the new channel roughness condition.

Table 9
Flow Distributions for New Channel Roughness

Event	Discharge, cfs	Natural Channel Discharge, cfs	Bypass Channel Discharge, cfs
1.2-year	1,500	1,500	0
10-year	7,100	3,600	3,500
20-year	9,000	4,100	4,900
100-year	14,600	5,700	8,900

Tailwater sensitivity

Flow distributions were documented for several tailwater elevations to determine the weir efficiency with respect to changes in tailwater elevation. The tailwater elevation in each channel (natural and bypass) was increased or decreased simultaneously by the amount indicated in the table. The resulting distributions are shown in Table 10.

Table 10
Flow Distribution for New Channel Roughness

Event	Discharge, cfs	Natural Channel Discharge, cfs	Bypass Channel Discharge, cfs	Tailwater
1.2-year	1,500	1,500	0	-1.0-ft
		1,500	0	**Design**
		1,500	0	+1.0-ft
		1,400	100	+2.0-ft
10-year	7,100	3,600	3,500	-1.0-ft
		3,600	3,500	**Design**
		3,450	3,650	+1.0-ft
		3,300	3,800	+2.0-ft
20-year	9,000	4,150	4,850	-1.0-ft
		4,100	4,900	**Design**
		4,000	5,000	+1.0-ft
		3,650	5,350	+2.0-ft
100-year	14,600	5,700	8,900	-1.0-ft
		5,700	8,900	**Design**
		5,800	8,800	+1.0-ft
		6,000	8,600	+2.0-ft

Water-surface profiles

Water-surface profiles were documented for the four flow events with four tailwater elevations. These data are presented in Table 11 and drawings in Appendix C.

Table 11
Water-Surface Elevations for New Channel Roughness

Station	Natural Channel Water-Surface Elevation, NGVD				Bypass Channel Water-Surface Elevation, NGVD			
	Event				Event			
	1.2-year	10-year	20-year	100-year	1.2-year	10-year	20-year	100-year
801+00	112.92	117.90	119.22	122.34				
800+00	113.04	118.23	119.49	122.40				
799+00	112.86	117.90	119.23	121.98				
798+00	112.50	117.60	119.01	121.74				
797+00	111.66	116.49	117.78	120.66				
796+00	110.22	112.47	112.92	115.11				
795+00	110.01	112.53	112.74	114.48				
794+00	109.98	112.98	113.52	115.02	No Flow			
793+00	109.89	113.28	113.85	115.38		109.11	110.64	114.42
792+00	109.14	112.56	113.85	115.95		109.08	110.73	114.75
791+00	108.84	111.78	112.71	114.90		108.93	110.46	114.21
790+00	108.69	111.66	112.74	114.93		108.57	110.07	113.79
789+00	108.42	111.18	112.02	114.21		108.60	109.98	113.76
788+00	108.12	110.61	111.69	114.06		108.60	110.16	113.94

Flow velocities

Velocities were higher for the new channel roughness condition. Since scour potential is based on velocity, this condition was documented. Flow velocities were measured in the vicinity of the weir and bridge pier to indicate the erosion potential of the channel. Velocities were also measured at the upstream and downstream extent of the model. Velocity measurement locations are shown in Figure 8.

Velocity data are shown in Plates 1 through 16.

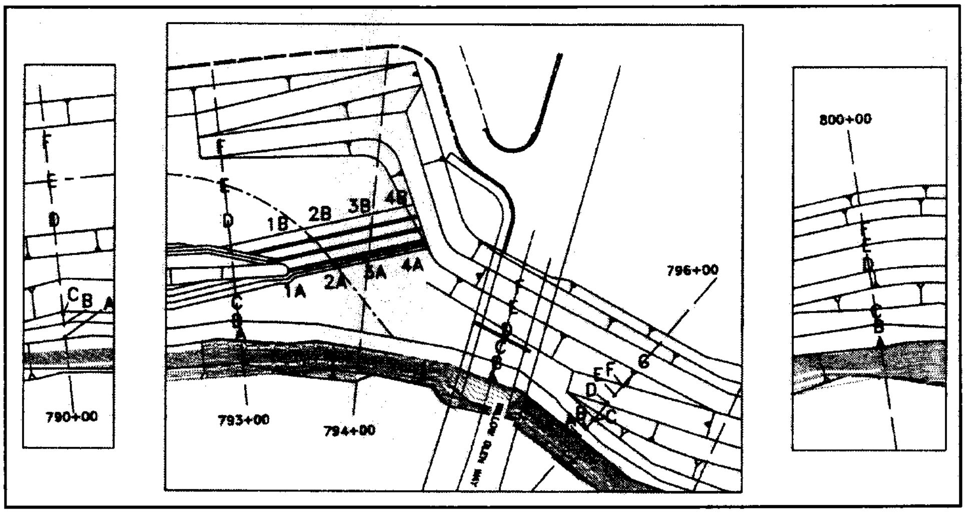

Figure 8. Velocity measurement locations

Sediment impact

Sediment was again introduced in the model to illustrate areas of likely deposition for the new channel roughness. The magnitude (quantity) of this deposition is not relative to prototype expectations. The procedure and material were the same as that used for the design roughness configuration. Figure 9 is a plan view of the model showing areas of sediment deposition.

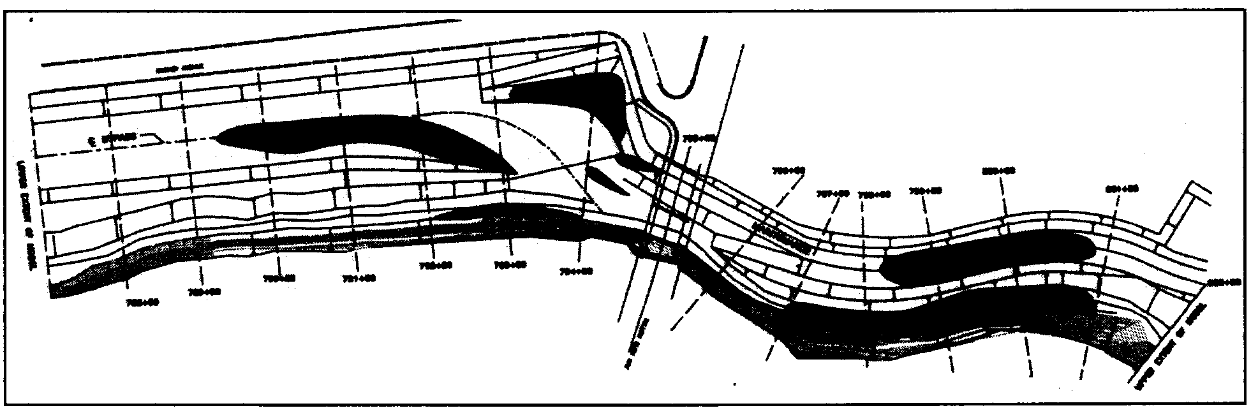

Figure 9. Sediment deposition new channel roughness

The flow distributions for the new channel roughness with and without sediment deposition are indicated in Table 12.

Table 12
Flow Distribution for New Channel Roughness with and without Sediment

Event	Total Discharge, cfs	Discharge Without Sediment Deposits		Discharge With Sediment Deposits	
		Natural Channel	Bypass Channel	Natural Channel	Bypass Channel
1.2 year	1,500	1,500	0	1,400	100
10 year	7,100	3,600	3,500	3,450	3,650
20 year	9,000	4,100	4,900	4,050	4,950
100 year	14,600	5,700	8,900	5,900	8,700

4 Discussion and Summary of Results

Flow Distribution

The flow distributions associated with the final weir design (Figure 6) met the initial requirements set by the San Francisco District. This is based on results for the channel representing the design roughness. The model results indicate a maximum of 1,500-cfs natural channel flow before the bypass channel is utilized. The flow distribution for the 100-year event is 5,000 cfs in the natural channel and 9,600 cfs in the bypass channel. These two distributions are indicative of a weir design that satisfies the original design requirements.

Tailwater Sensitivity

Flow distributions were recorded for each event with changes in tailwater elevation. These elevations were: the design; design – 1.0 ft; design + 1.0 ft; and design + 2.0 ft. While there were some changes in distribution resulting from changes in tailwater elevation, they are considered to be small variations. The maximum distribution variation was 600 cfs for the 10-year event with the design roughness and 450 cfs for the 20-year event with the new channel roughness. This small change in distribution with respect to tailwater change was attributed to the weir's location. It is located at the downstream extent of a supercritical zone near the Willow Glenn Way bridge. The water-surface elevation in a region of supercritical flow is not controlled by tailwater, and therefore is not responsive to these tailwater elevation changes.

The left end of the weir (end closest to the natural channel) is near a jump area (transitioning from supercritical to subcritical flow) in the natural channel. This affected area of the weir is responsible for the small changes in distribution. If the weir were located entirely in the supercritical flow, no changes in distribution would be expected due to tailwater changes unless the tailwater changes were large enough to change the flow regime at the weir.

Water-Surface Profiles

The water-surface elevation was measured for each event with the four-tailwater elevation previously described and with the two different channel roughnesses. The changes were significant, but the river discharge was contained within the channels. The only exception was for the 100-year event with the design + 2.0-ft tailwater elevation. During this event the natural channel overtopped on the right bank and spilled into the bypass channel. No overtopping occurred on the landside of either channel, thus indicating proper design of channel dimensions to prevent overtopping.

Flow Velocities

Flow velocities were considered to be most critical or at their highest with the new channel (smoothest) roughness (Table 1). Therefore, channel velocities were measured in the supercritical zone near the bridge pier and the weir for the new channel roughness configuration. The highest observed velocities were for the 100-year event. These velocities reached as high as 20-fps near the weir and 26.5-fps under the bridge. At these velocities, channel protection would be required to prevent channel erosion.

Sediment Impact

The sediment used in the model study was not a representation of the prototype sediment material. The model material was much larger and therefore could not be used to indicate quantities of deposition. This portion of the study is primarily for the demonstration of areas in the channels where sediment will likely deposit. The areas of deposition are shown in Figures 7 and 9.

The flow distributions with and without sediment tabulated in Table 6 show that the changes in flow distribution for the 100-year event is insignificant (approximately 0.5 percent of the total flow).

Willow Glenn Way Bridge

The Willow Glenn Way bridge pier (single pier) alignment had a significant impact on the distribution of flow between the two channels. The pier design and location was provided by the San Francisco District. The pier orientation was determined in the model. It was oriented to be streamlined in the flow, thus reducing its influence in the flow field. This orientation provided the least impact on flow distributions.

The pier was moved across the channel up to 10 ft toward the right bank and up to 10 ft toward the left bank. These location adjustments did not impact the flow distributions as long as the pier orientation (intersect angle with the weir) was not changed. Changes in the pier orientation will direct flow either into or away from the weir thus changing its performance. Figure 10 shows the bridge pier as located in the model.

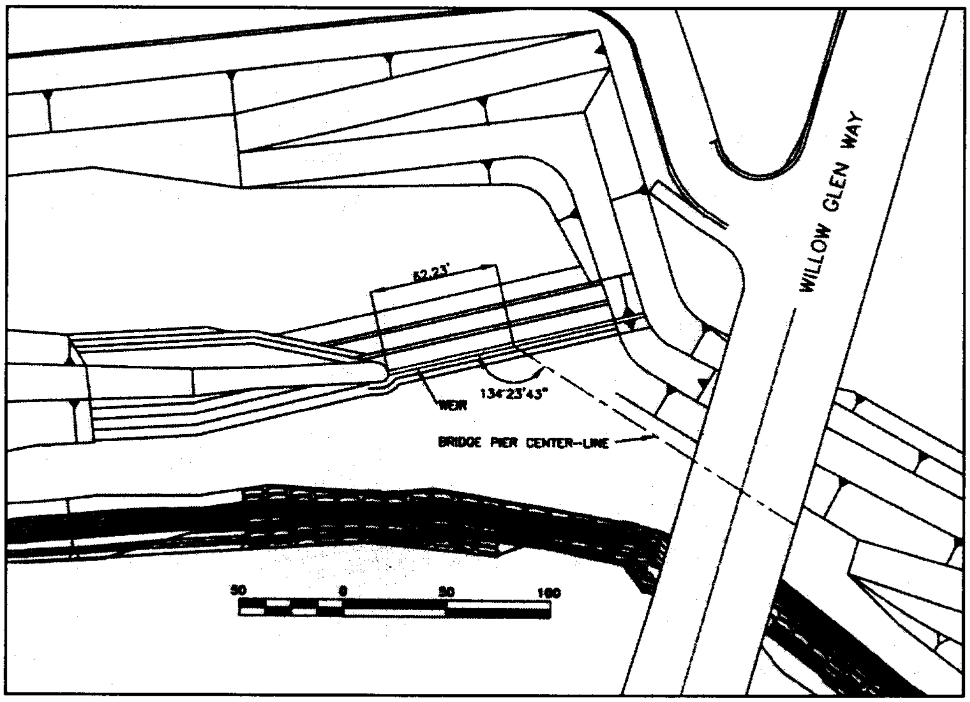

Figure 10. Upper Guadalupe bridge pier center-line location Willow Glen Way

To determine the bridge freeboard, the model bridge pier was marked to show the lowest soffit elevation of 123.27.[1] Freeboard measurements were measured with and without debris buildup on the pier. Model debris consisted of a packing material commonly known as rubberized hair. The rubberized hair was sized to simulate approximately 8-ft wide debris buildup. The freeboard was measured from the mark at el 123.27 to the water surface. The data are shown in Table 13. As mentioned in the report, the bridge pier is located in a supercritical flow regime. The debris buildup prevented "ride-up" of flow on the pier and increased the freeboard.

Table 13
Willow Glen Way Bridge Freeboard, 100-Year Event, with and without Debris on Pier

Tailwater	Freeboard without Debris, ft	Freeboard with Debris, ft
-1.0'	2.1	7.5
Design	2.1	6.9
+1.0'	2.1	6.9
+2.0'	2.1	6.9

[1] All elevations (el) cited herein are in feet as referred to in the National Geodetic Vertical Datum (NGVD) of 1929. To convert feet to meters, multiply by 0.3048.

5 Conclusions and Recommendations

Weir Design

The final weir design (Figure 6) will provide flow distributions that meet the original design requirements. The design requirements, as stated by the San Francisco District, were to pass a 1.2-year event (1,500 cfs) in the natural channel and to distribute the 100-year event (14,600 cfs) with 5,000 cfs in the natural channel and 4,600 cfs in the bypass channel.

This weir design and location does not appear to be sensitive to tailwater elevation changes ranging from the design tailwater elevation –1.0 ft to the design tailwater elevation +2.0 ft. Nor does this weir configuration appear sensitive to channel sedimentation (as conducted in the study).

The weir performance is sensitive to the bridge pier (Willow Glen Way bridge) alignment. However, this study indicates that the pier location can be moved plus or minus 10 ft (left or right) without affecting the weir performance.

Any deviations in weir or bridge pier design should be model studied to insure the weir performance is not changed.

Flow Velocities

The velocities in the natural channel near the bridge and in both channels near the weir are in excess of 20 fps. If the San Francisco District determines these velocities will erode the channel material, steps should be taken to stabilize the channel. At these high velocities, large diameter riprap or a concrete line channel may be necessary.

If the channel is not stabilized, grade control structures should be used to maintain the channel geometry. If the channel geometry is allowed to change, the flow regime (supercritical and subcritical flow) will change and require additional model evaluation to ensure proper weir performance.

References

Hite, J. E., Jr., (1998). "Guadalupe River and bypass culvert, San Jose, California," TR CHL-98-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

King, H. W., and Brater, E. F. (1976). *Handbook of hydraulics*. McGraw-Hill, Inc.

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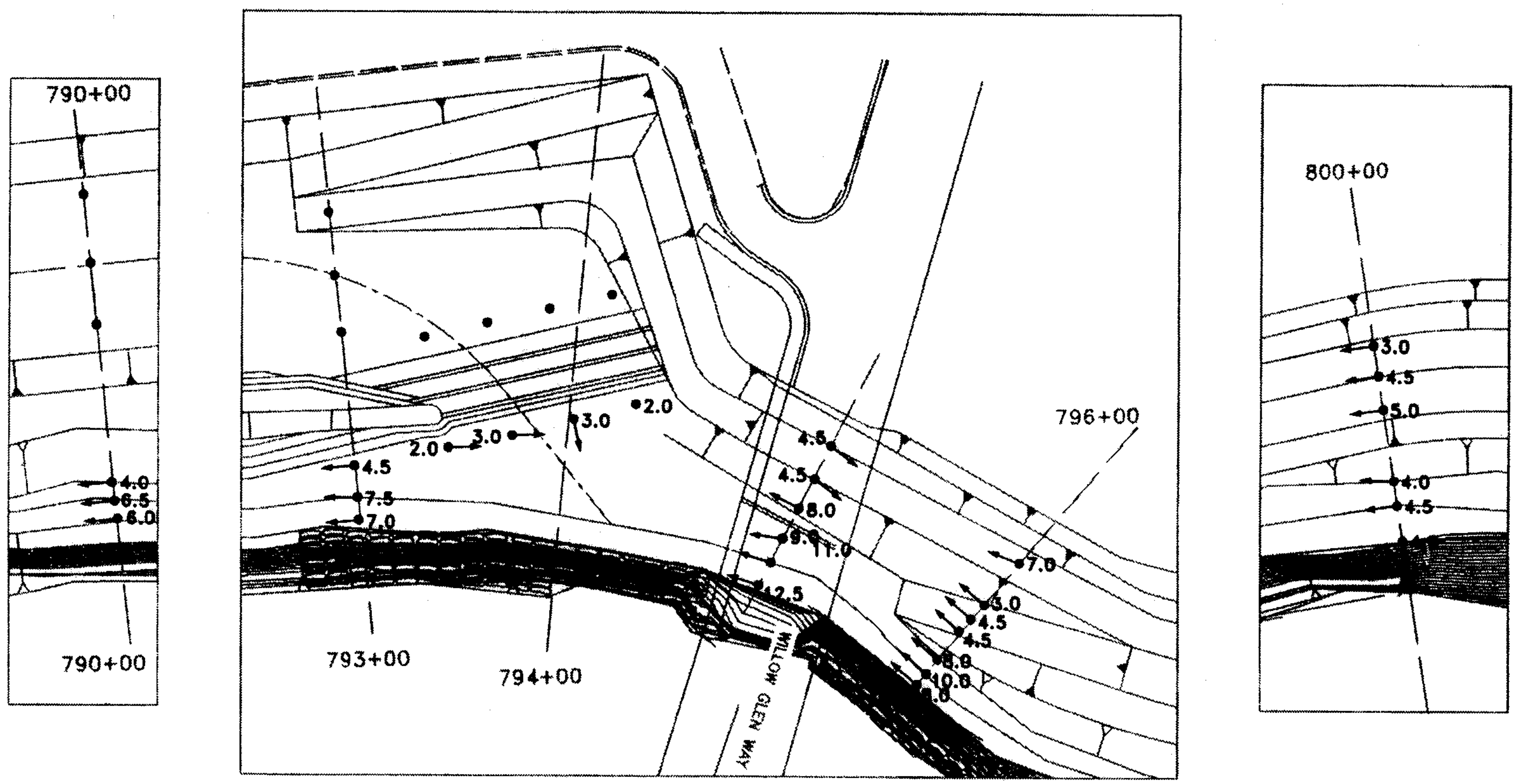


NOTE: VELOCITIES ARE IN FEET PER SECOND

CHANNEL VELOCITIES

FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
1.2 YEAR EVENT
DESIGN STAGE

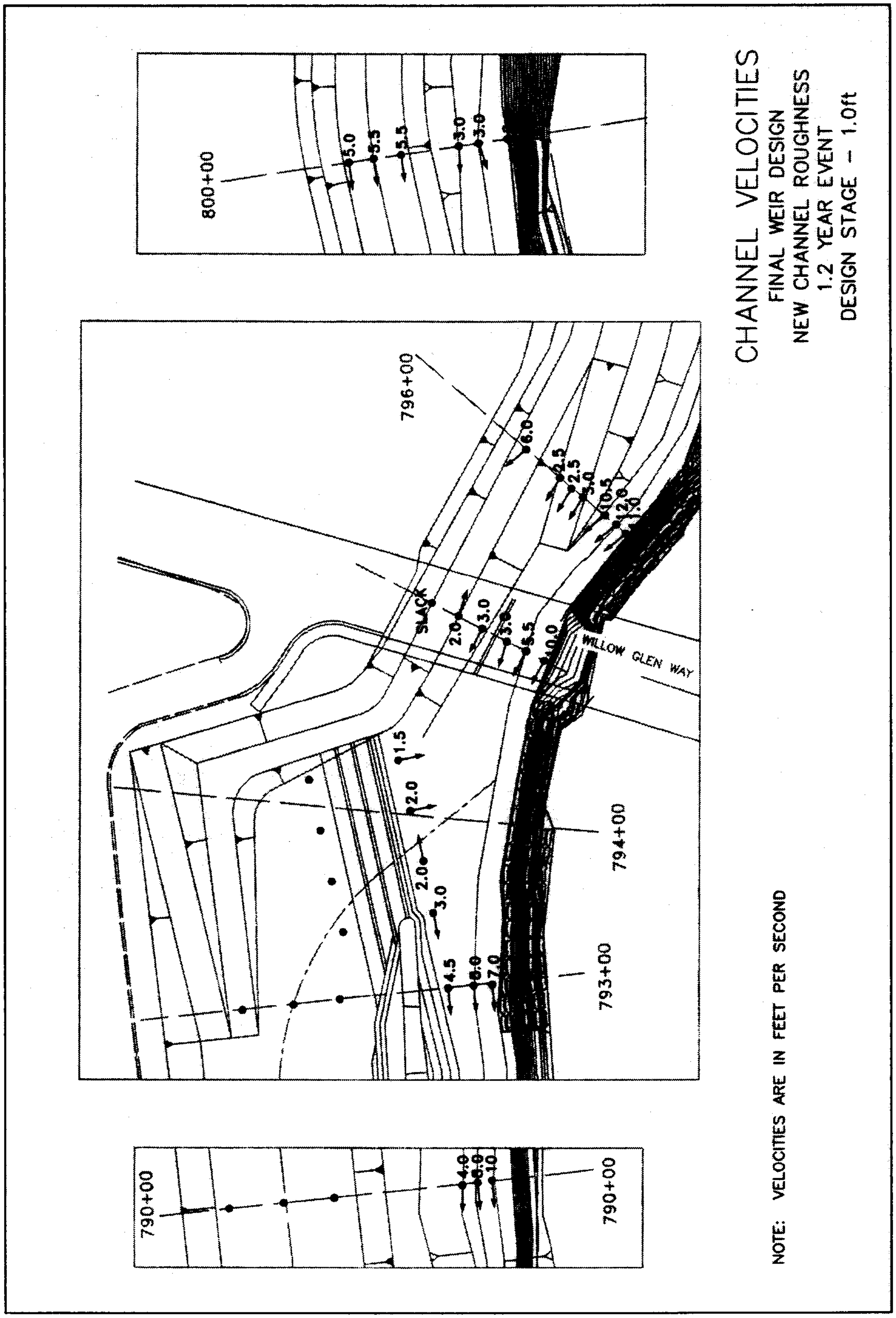
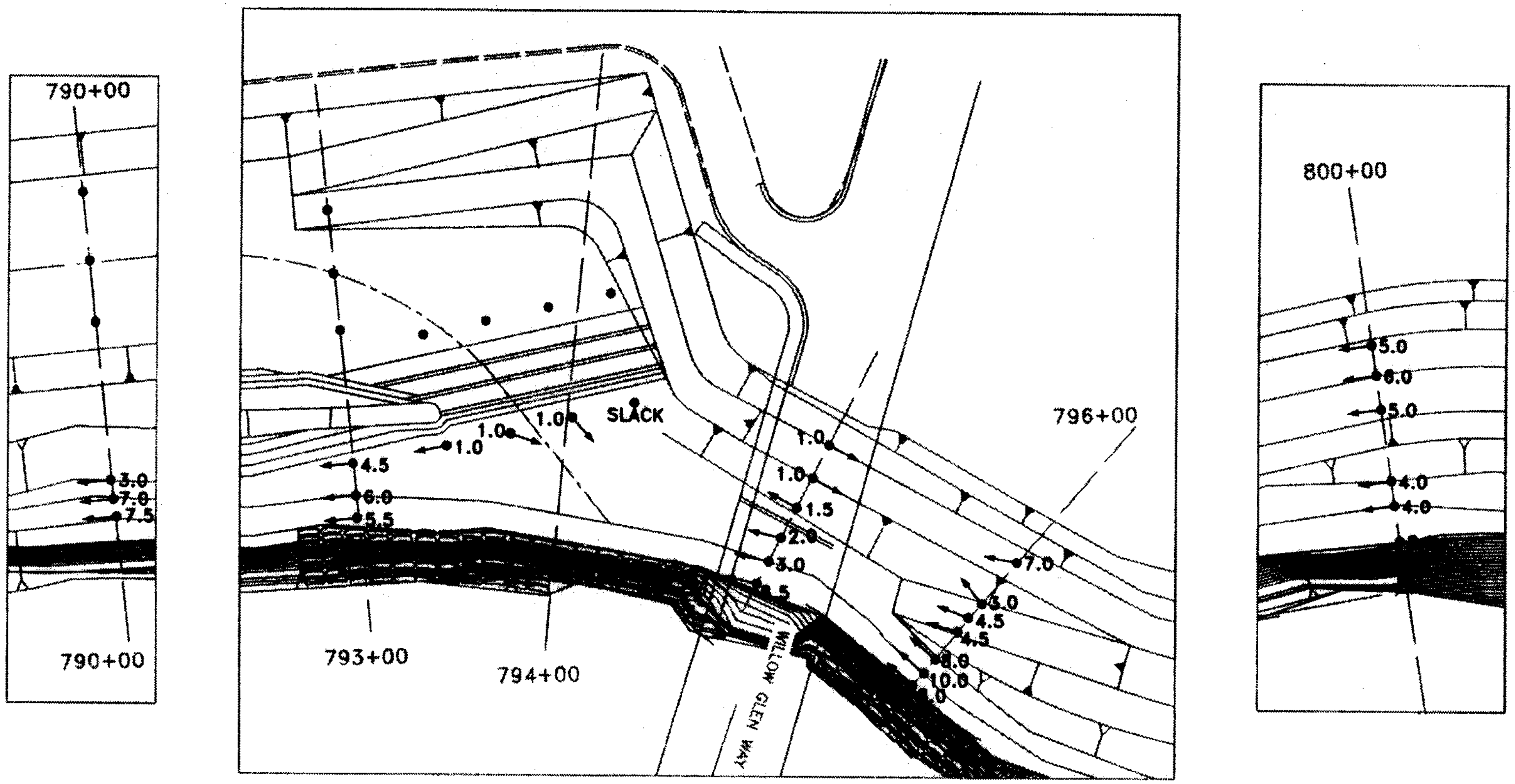
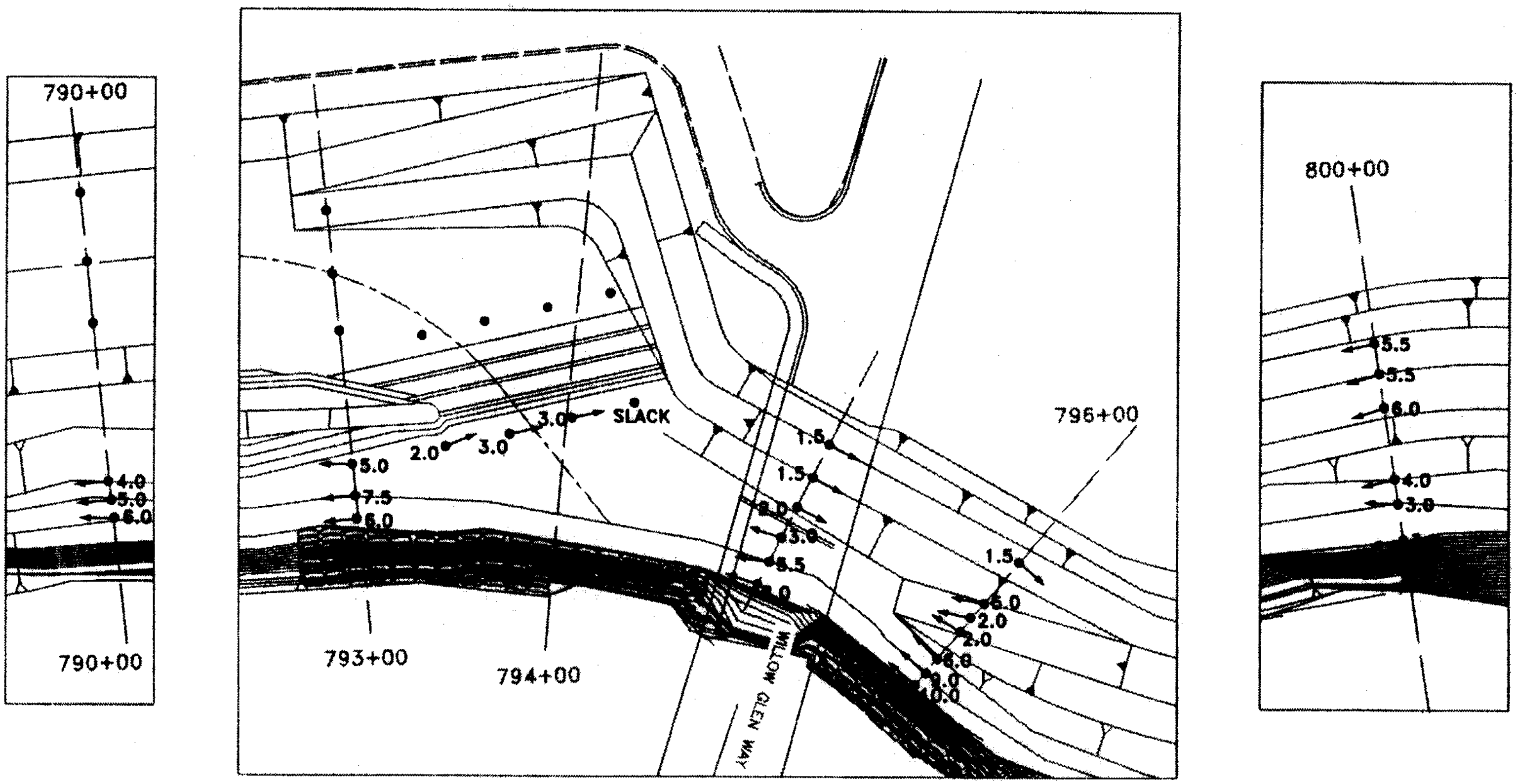

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
1.2 YEAR EVENT
DESIGN STAGE – 1.0ft
NOTE: VELOCITIES ARE IN FEET PER SECOND
800+00
796+00
794+00
793+00
790+00
WILLOW GLEN WAY
SLACK

Plate 2



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
1.2 YEAR EVENT
DESIGN STAGE + 1.0ft

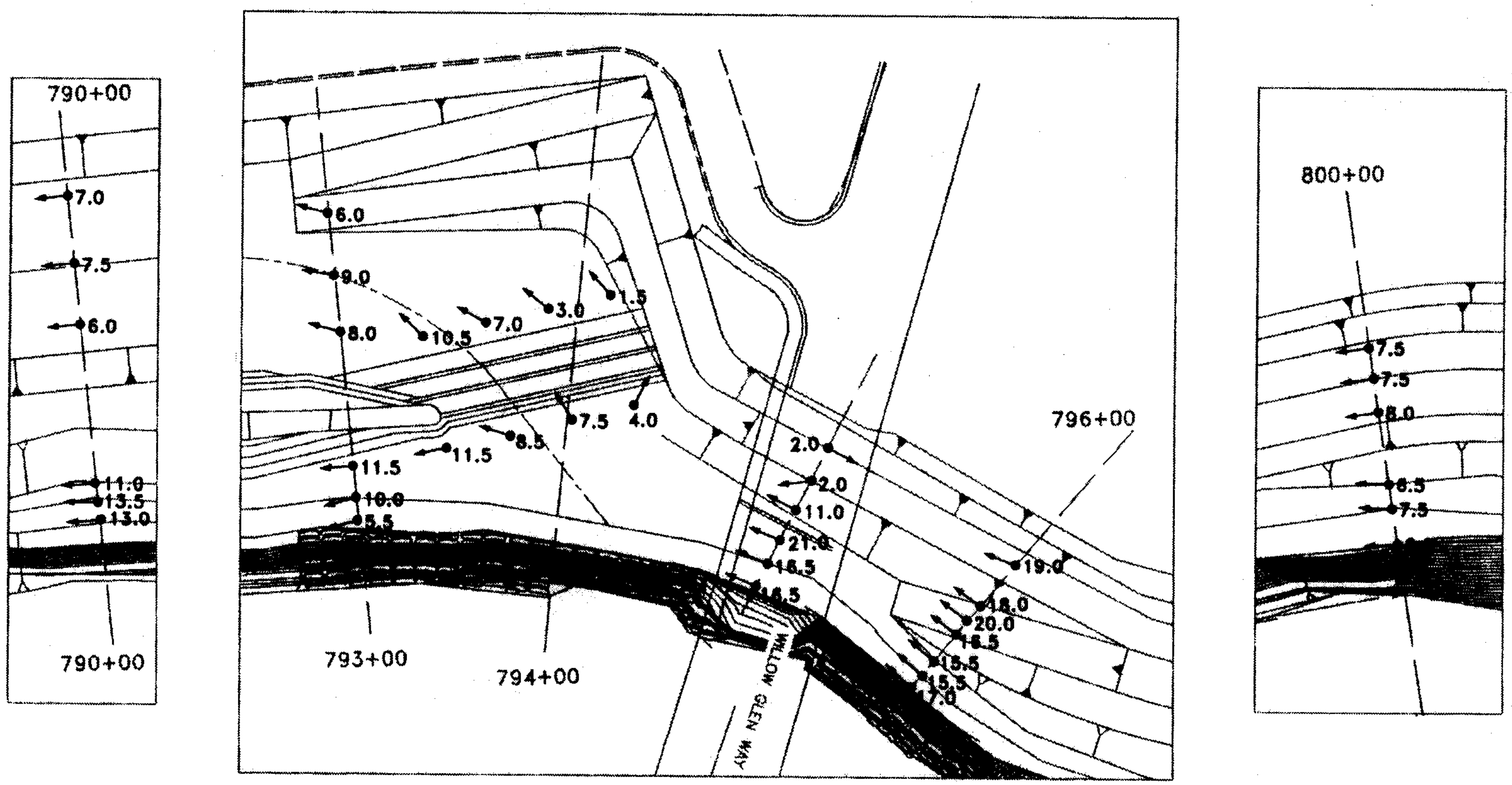


NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES

FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
1.2 YEAR EVENT
DESIGN STAGE + 2.0ft

Plate 4



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
10 YEAR EVENT
DESIGN STAGE

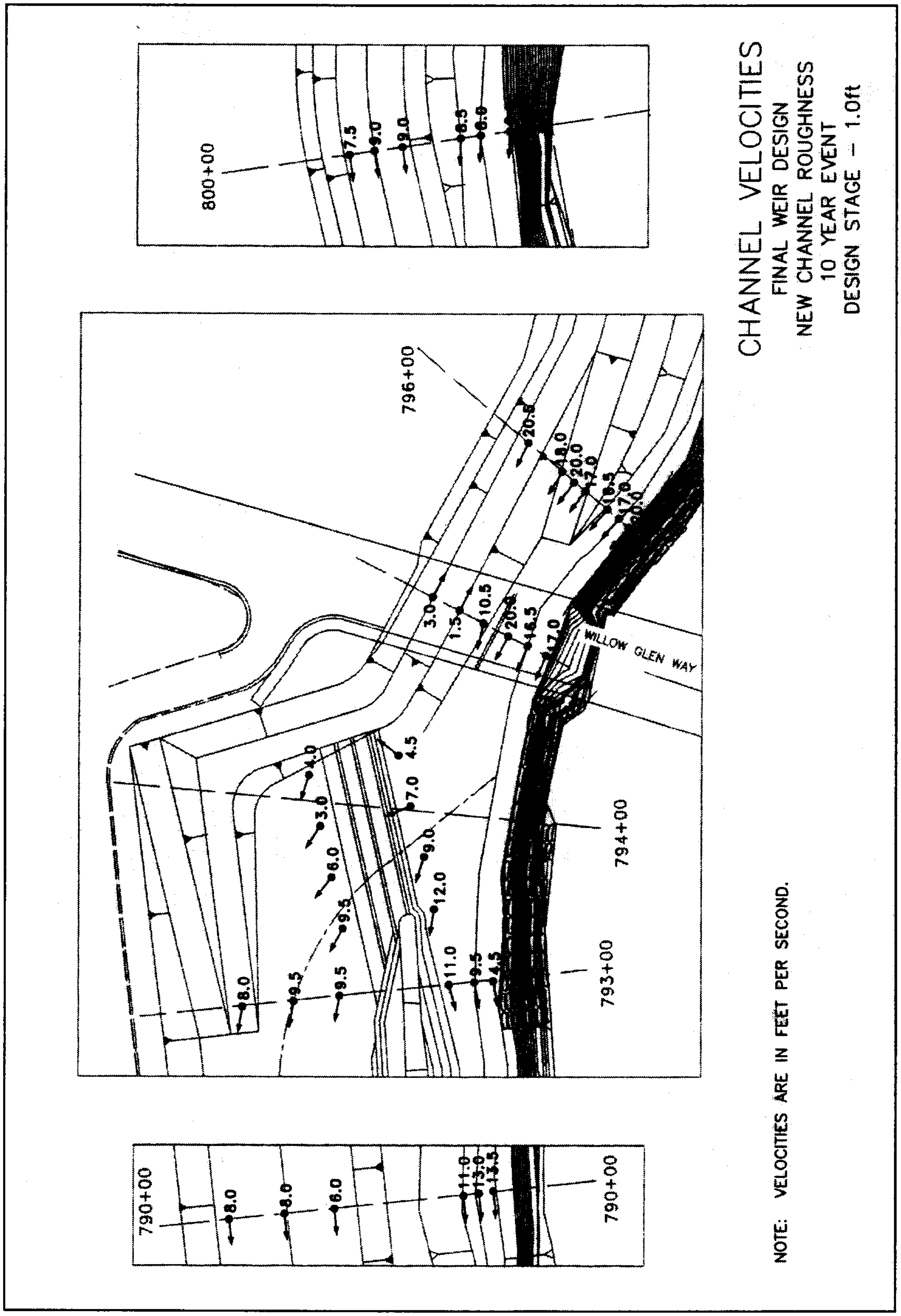
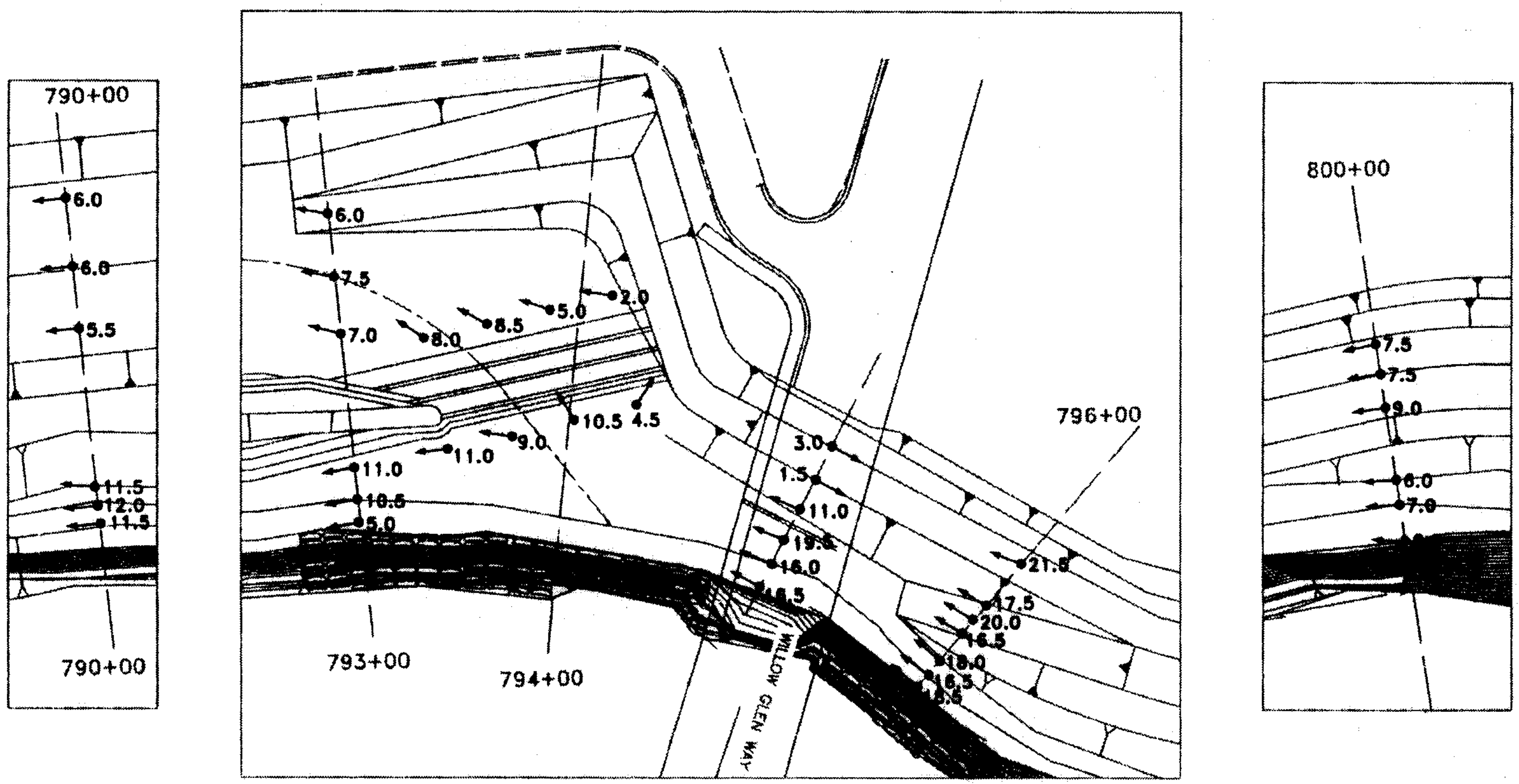

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
10 YEAR EVENT
DESIGN STAGE – 1.0ft
NOTE: VELOCITIES ARE IN FEET PER SECOND.
WILLOW GLEN WAY
800+00
796+00
794+00
793+00
790+00

Plate 6



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
10 YEAR EVENT
DESIGN STAGE + 1.0ft

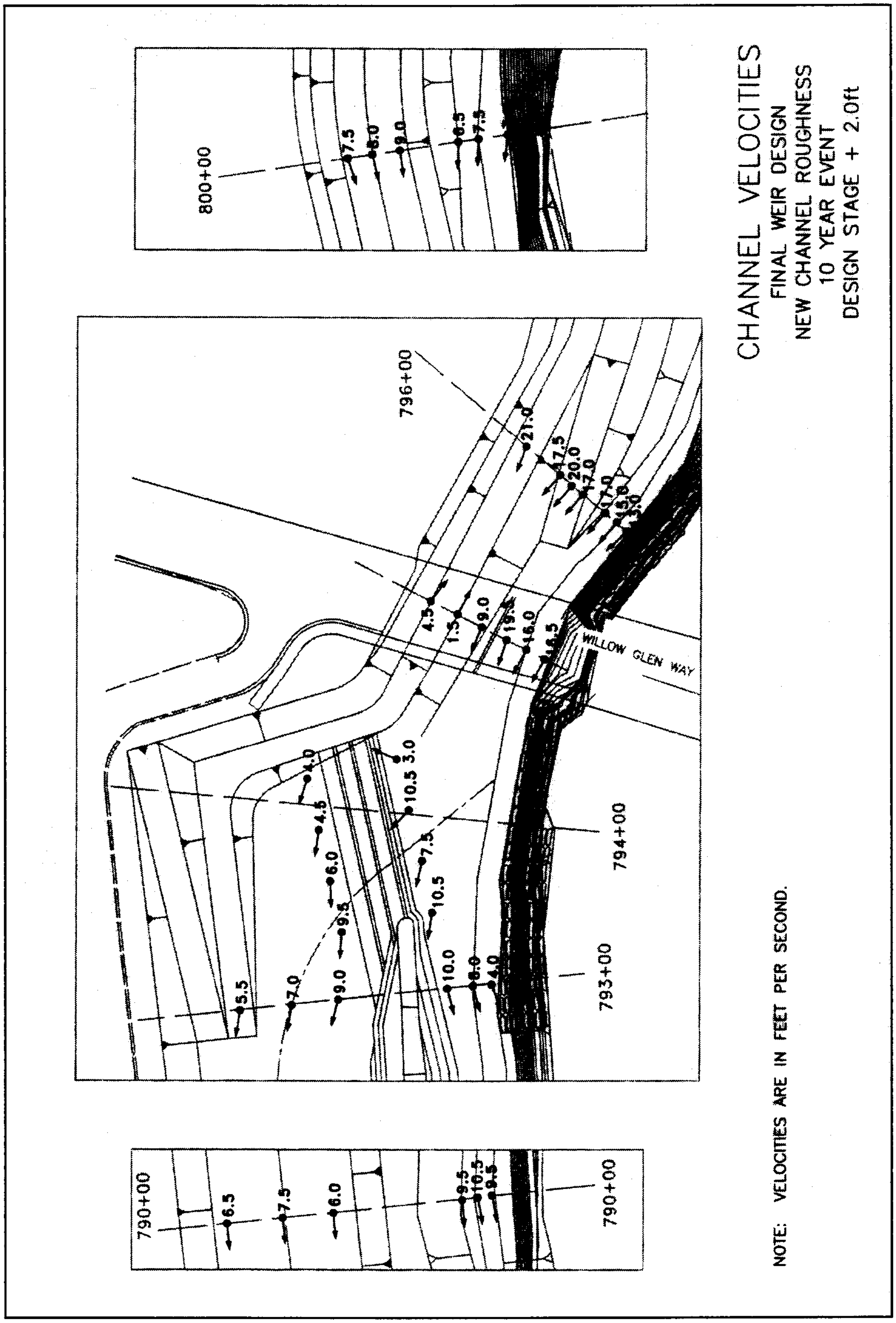
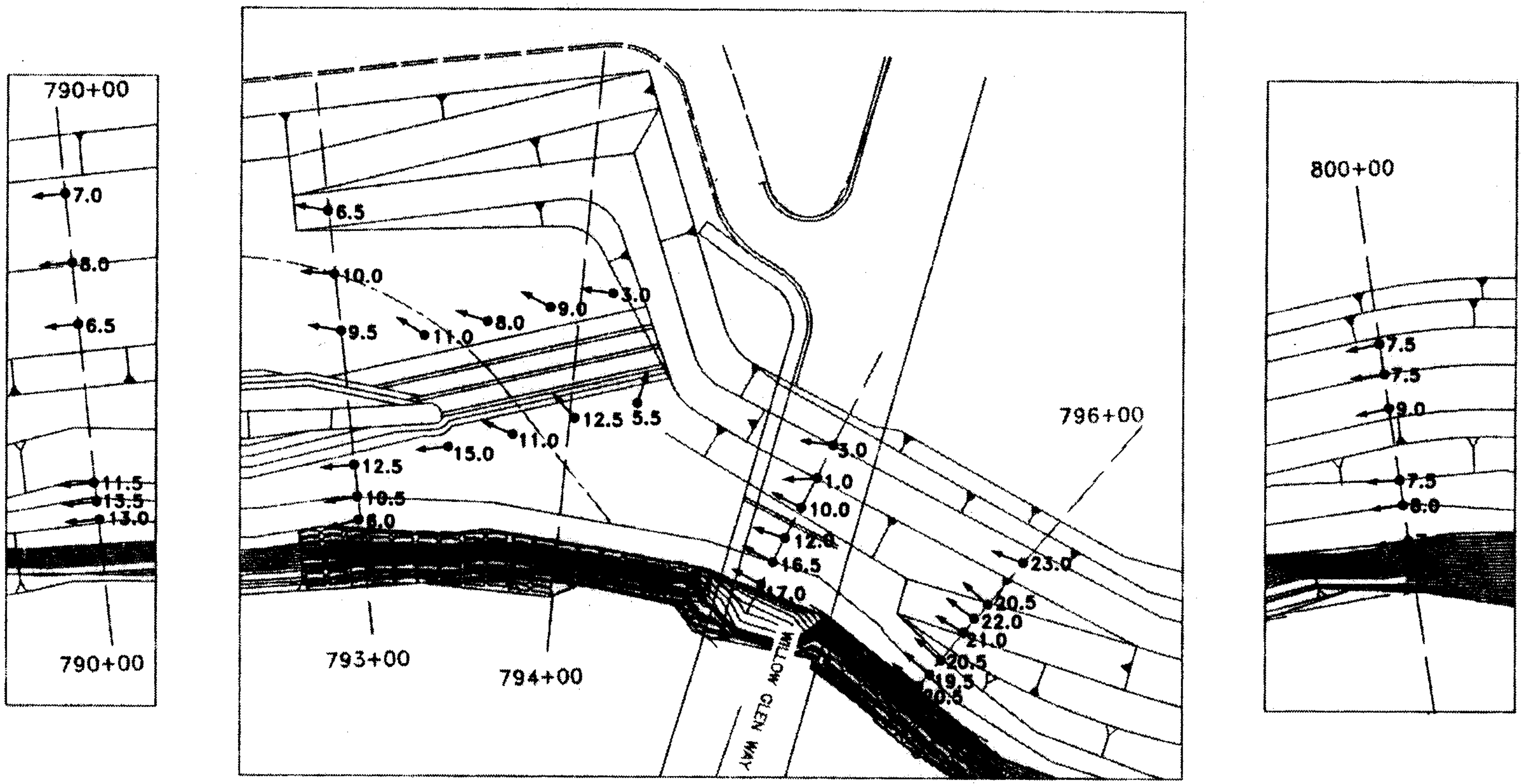

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
10 YEAR EVENT
DESIGN STAGE + 2.0ft
NOTE: VELOCITIES ARE IN FEET PER SECOND.
WILLOW GLEN WAY
800+00
796+00
794+00
793+00
790+00

Plate 8



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
20 YEAR EVENT
DESIGN STAGE

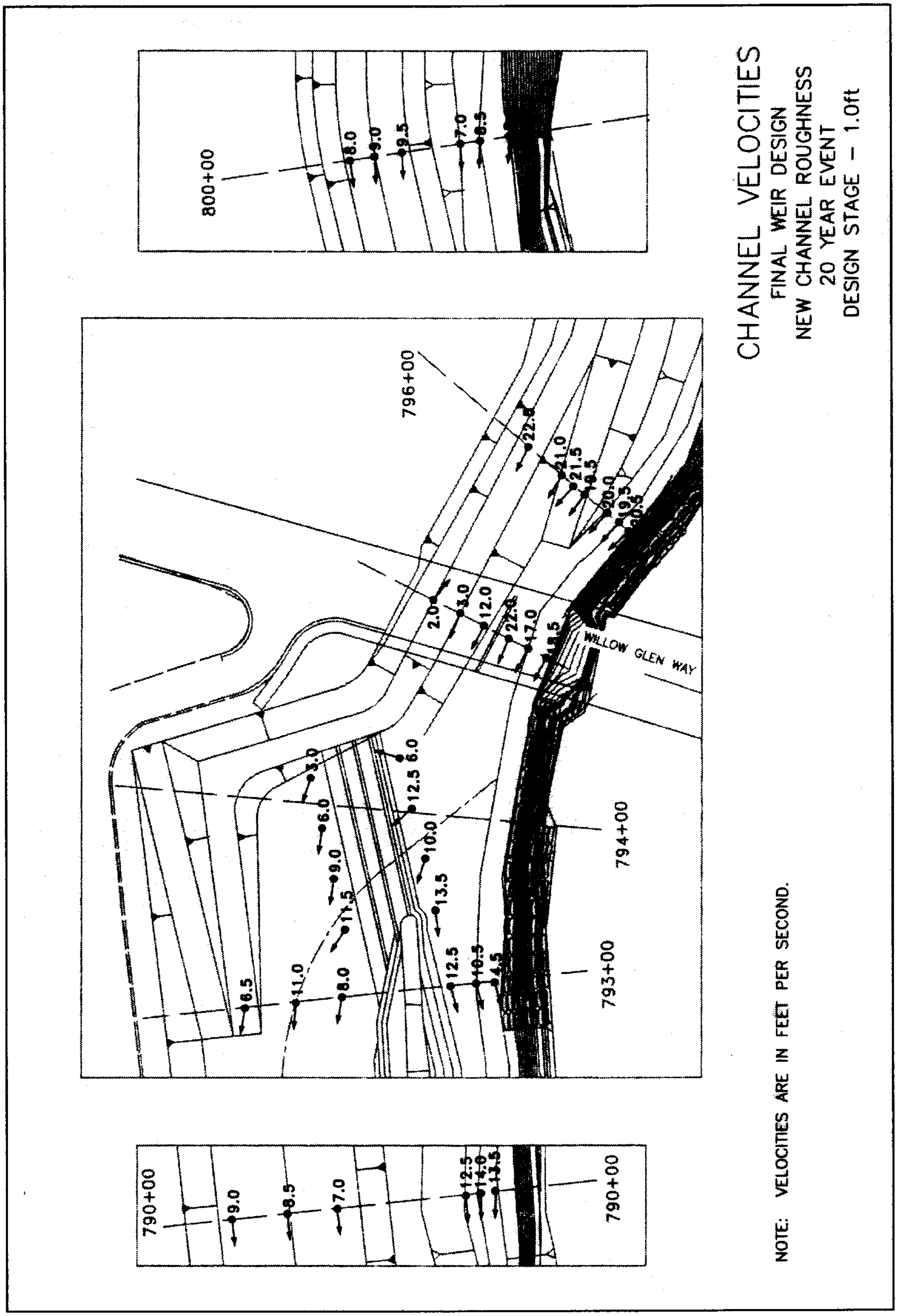
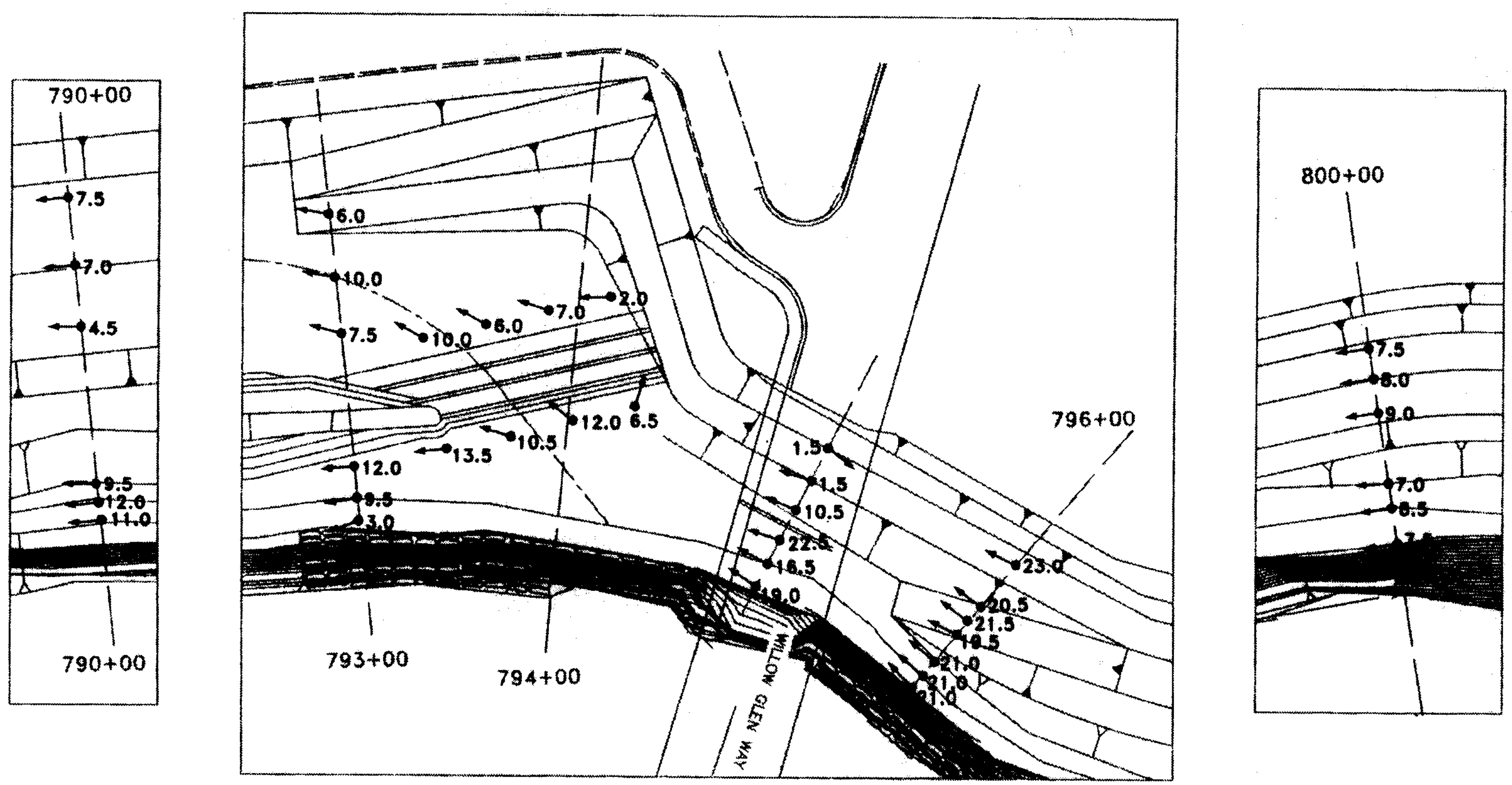

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
20 YEAR EVENT
DESIGN STAGE – 1.0ft
NOTE: VELOCITIES ARE IN FEET PER SECOND.
WILLOW GLEN WAY
800+00
796+00
794+00
793+00
790+00
790+00

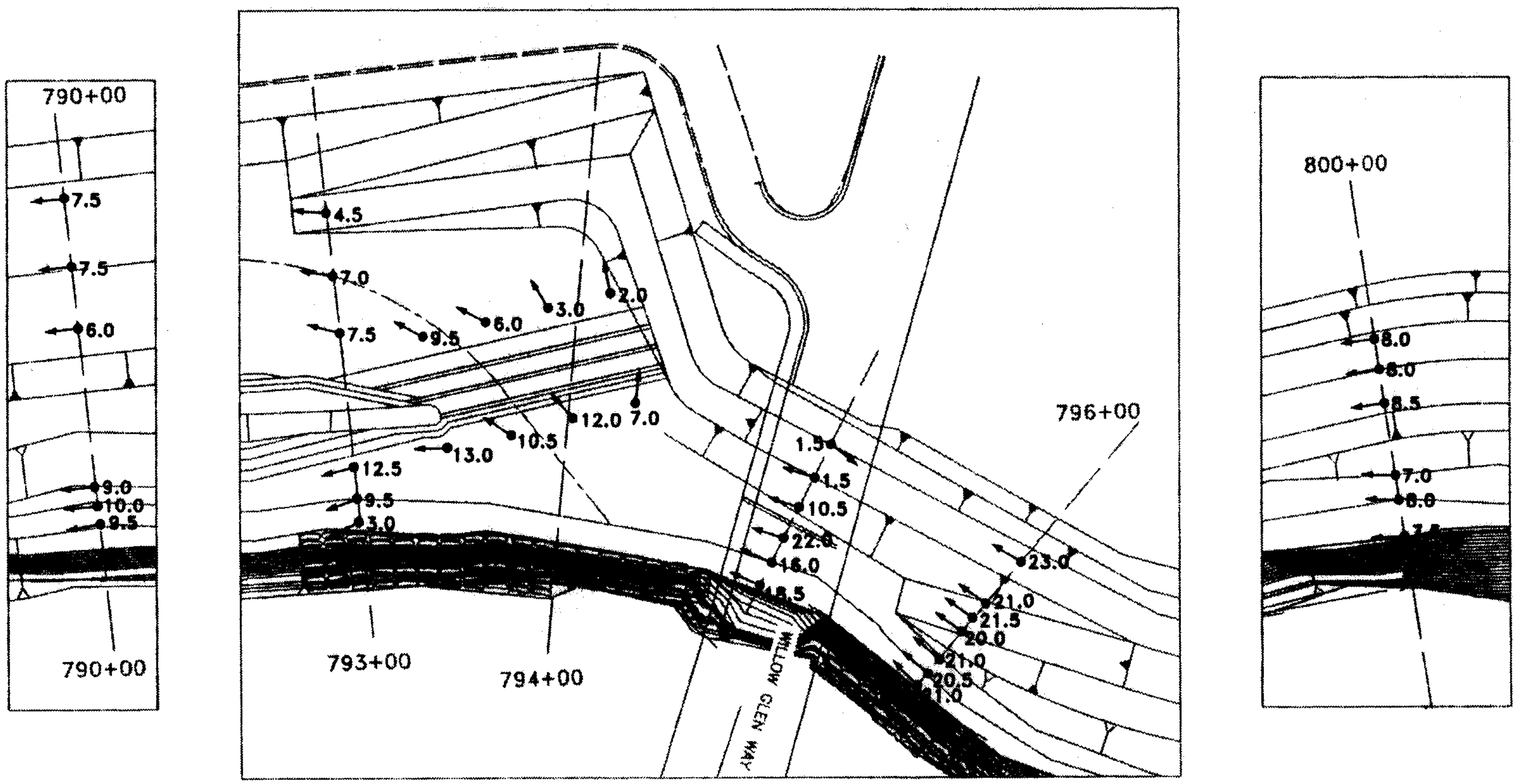
Plate 10



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES

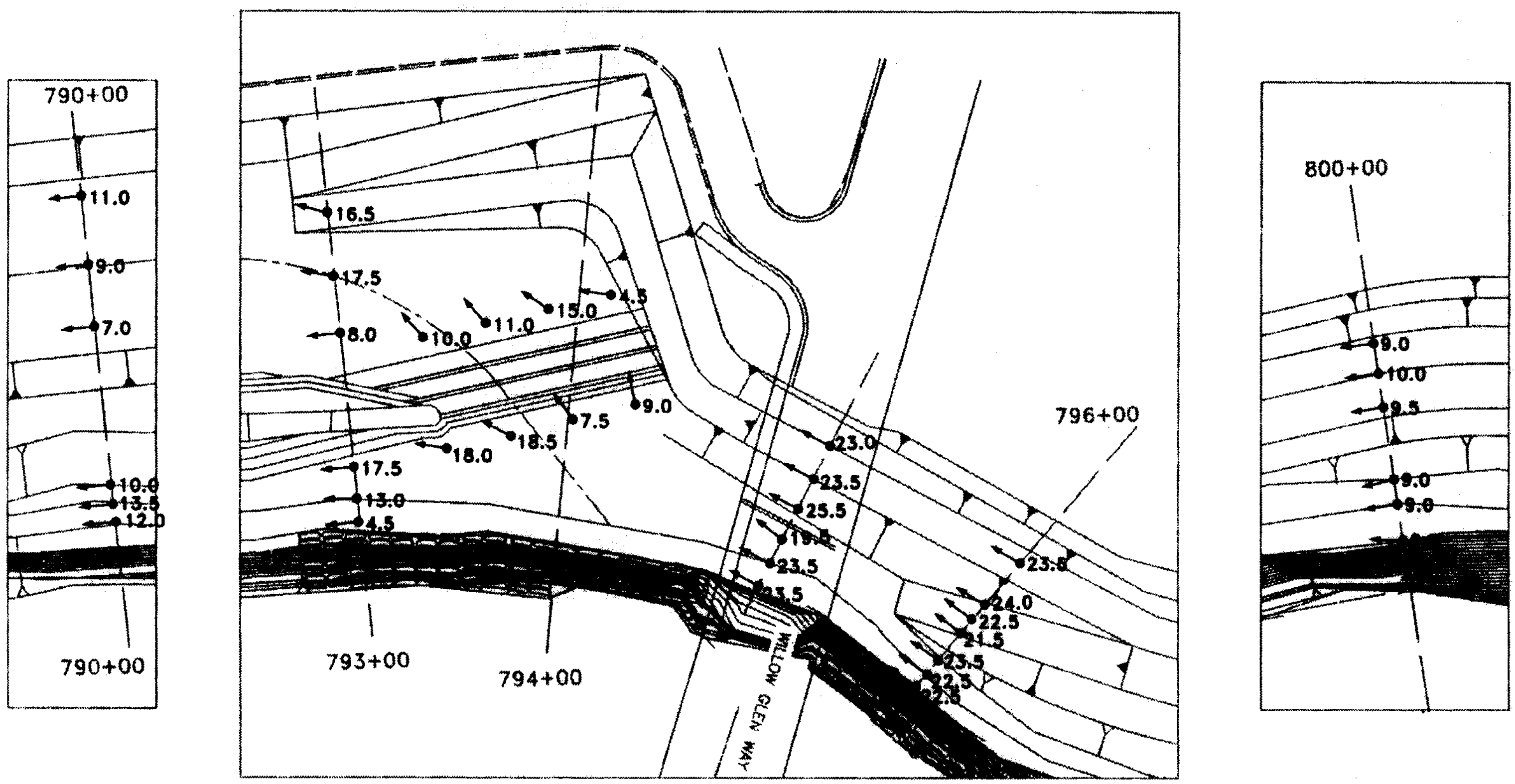
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
20 YEAR EVENT
DESIGN STAGE + 1.0ft



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
20 YEAR EVENT
DESIGN STAGE + 2.0ft

Plate 12

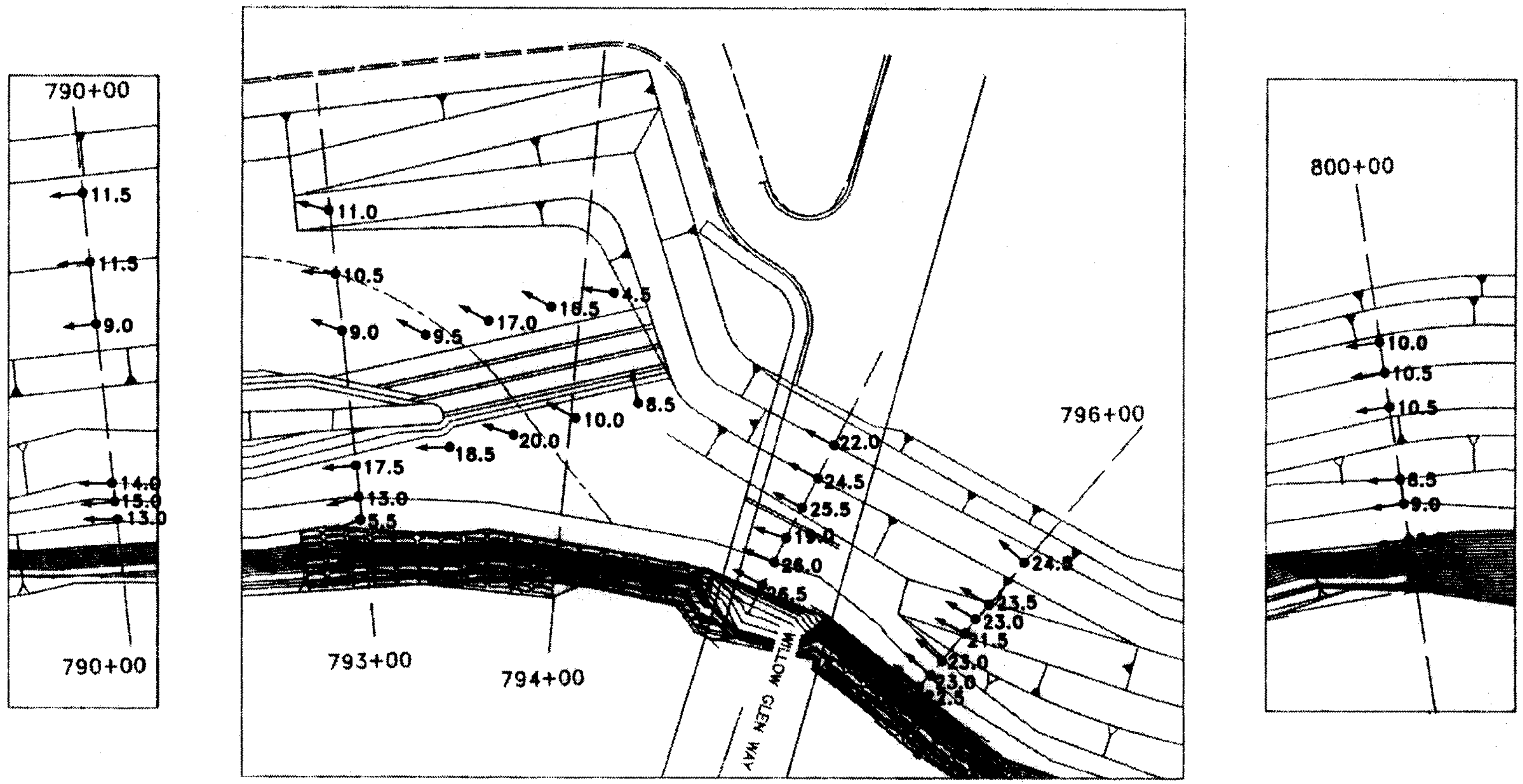


CHANNEL VELOCITIES

FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
100 YEAR EVENT
DESIGN STAGE

NOTE: VELOCITIES ARE IN FEET PER SECOND.

Plate 13

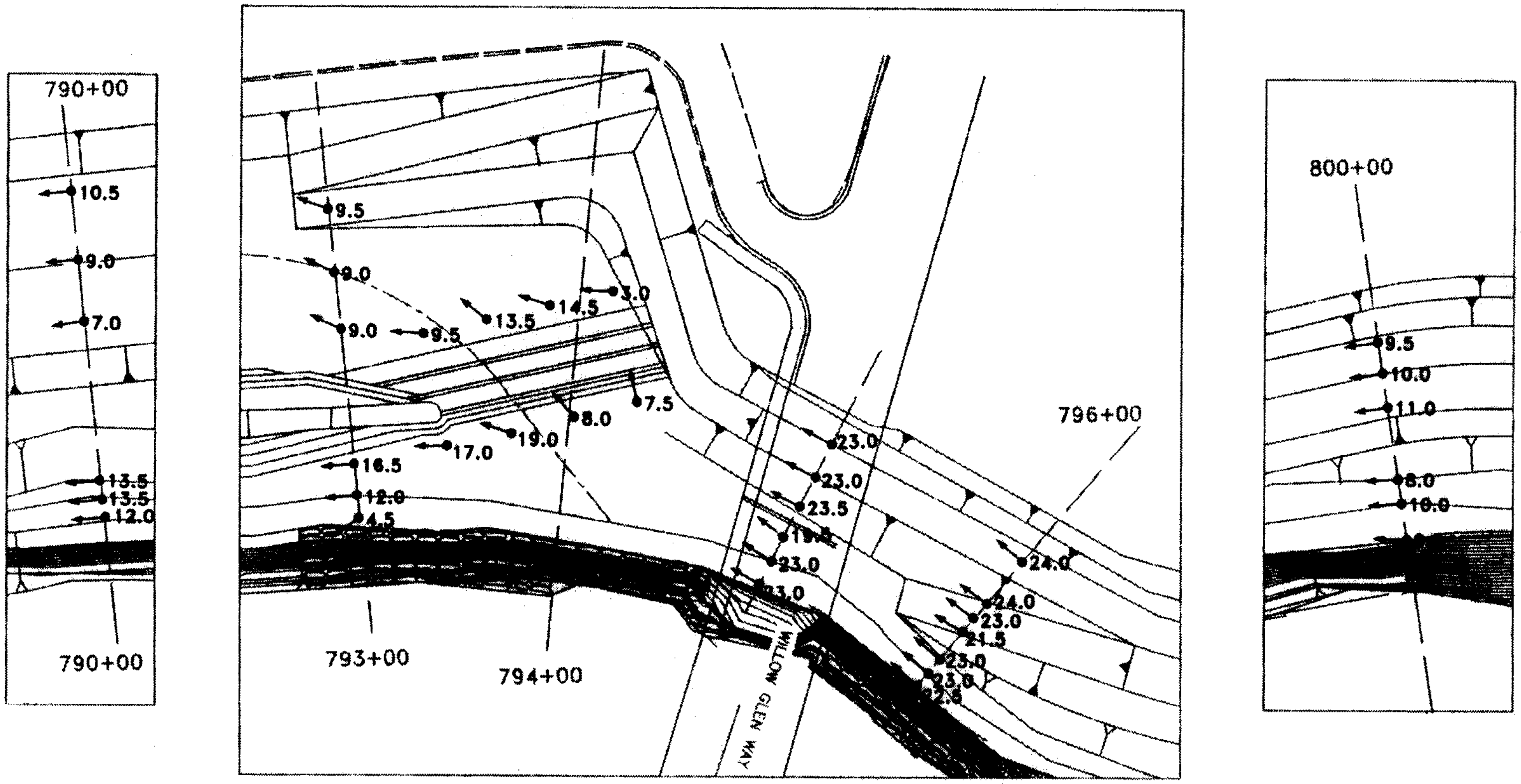


NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES

FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
100 YEAR EVENT
DESIGN STAGE – 1.0ft

Plate 14



NOTE: VELOCITIES ARE IN FEET PER SECOND.

CHANNEL VELOCITIES

FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
100 YEAR EVENT
DESIGN STAGE + 1.0ft

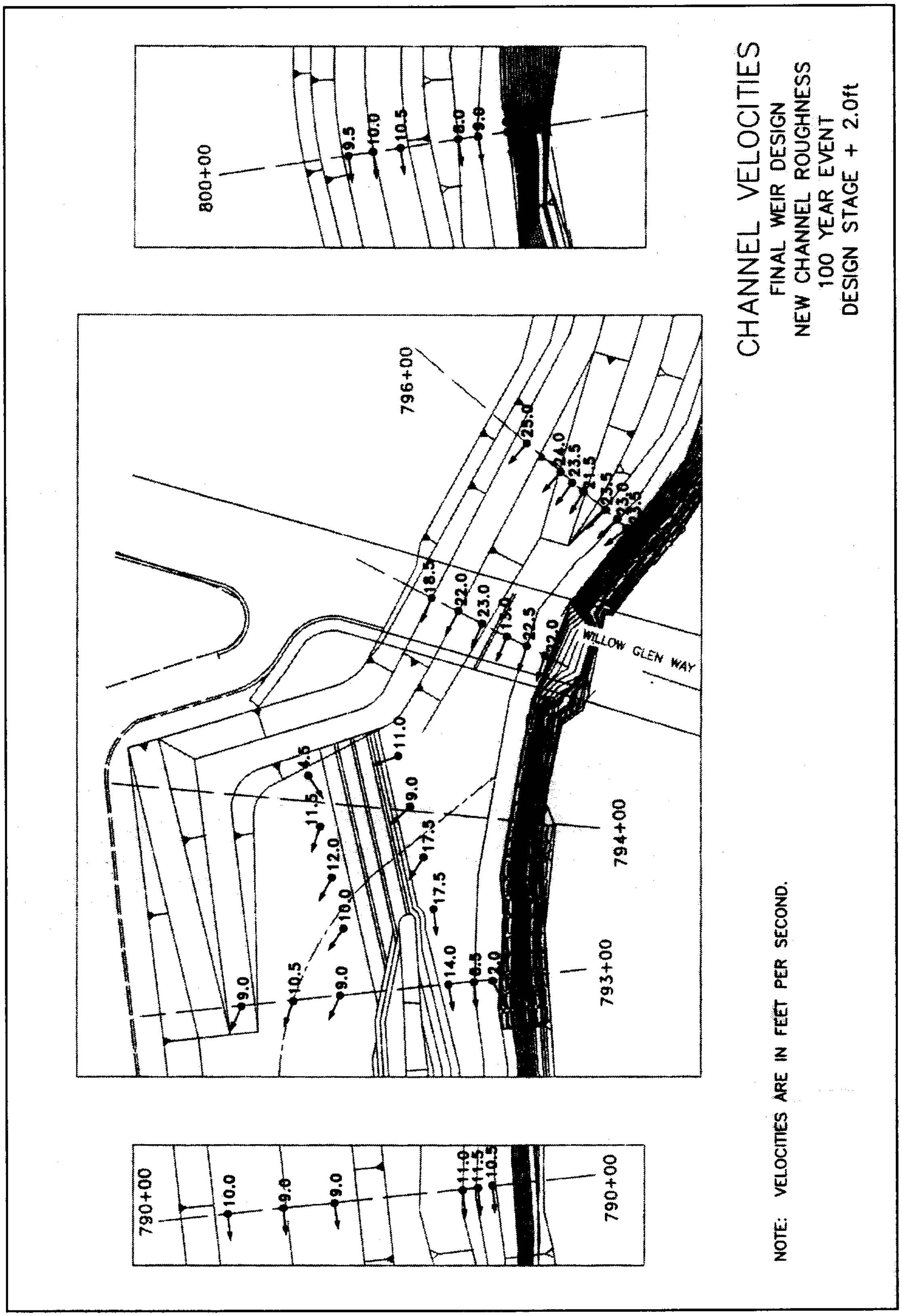

CHANNEL VELOCITIES
FINAL WEIR DESIGN
NEW CHANNEL ROUGHNESS
100 YEAR EVENT
DESIGN STAGE + 2.0ft
NOTE: VELOCITIES ARE IN FEET PER SECOND.
800+00
796+00
794+00
793+00
790+00
WILLOW GLEN WAY

Plate 16

Appendix A
Weir Coefficient Calculations

To: Mr. Randy Oswalt

From: Dwayne Fuller

Date: 3/18/02

Re: Upper Guadalupe Model Study, Weir Coefficient for Model Discharge

---

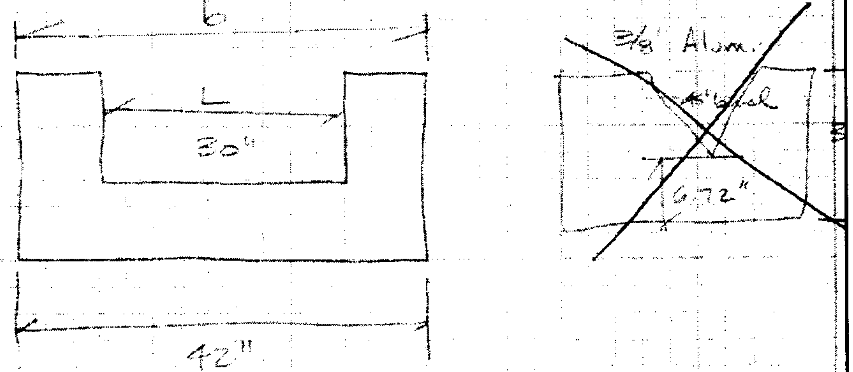
Randy,

This memo is intended to discuss the method used to set the discharge in the subject model. We are using two "horizontal sharp crest" weirs, one each downstream of the natural channel and the bypass channel. I used the discharge coefficient, Ce = 3.2, and assumed the H/P was close enough to 0 to be negligible. These two values were used to set the weir sizes (see Figure 1) and used to measure the discharges in the model.

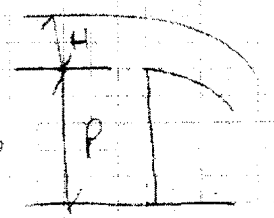
As a result of Dr. Wang questioning my selection of a discharge coefficient during our meeting and model demonstration earlier this week, I took another look at the discharge coefficient and H/P ratio. I selected Ce values based on the head over the weir for both the natural and bypass channels for the four events used in the model evaluations. I used the method described in "Handbook of Hydraulics" written by Brater and King (see Figures 2-5).

For all seven discharge settings (four in the natural channel and three in the bypass channel) the discharge coefficient was 3.2.

11/2000



Horizontal Crest Weir



$Q = C L H^{3/2}$ eq. 5.10, King's

for $\frac{H}{P}$ very small ⟸ $H \ll P$ ∴ $\frac{H}{P}$

$C_e \cong \underline{\underline{3.2}}$

$L_e = L + K_L$ eq. 5.35

$\frac{L}{b} = \frac{30}{50} = 0.6$

$K_L = 0.012$ ft

$L_e = 2.5' + 0.012' = \underline{2.512}'$

$H^{3/2} = \frac{Q}{C L_e}$

$H = \left(\frac{Q}{C L_e}\right)^{2/3}$; $Q_{max} = 9600$ cfs ⇒ 1.947 cfs

$Q_{min} = 1500$ cfs ⇒ 0.304 cfs

$H_{max} = \left(\frac{1.947}{3.2(2.512)}\right)^{2/3} = .389'$ or $\boxed{4.66''}$ $H_{min} = \left(\frac{0.304}{3.2(2.512)}\right)^{2/3} = .113'$ or $\boxed{1.35''}$

Figure A1. Weir coefficient calculations

Natural Channel

$\frac{L}{b} = \frac{2.340'}{4'} = 0.585$ $\qquad P = 0.958'$

$K_L = +0.012$, Fig 5-3

$L_e = 2.340' + 0.012 = \underline{2.352}'$

<u>1.2 yr event</u>

1500 cfs $\quad H \cong 0.118' \rightarrow$ based on $C_e = 3.2$ & $H/P = 0$

$\frac{H}{P} = \frac{0.118'}{0.958'} = \underline{0.123}$

$C_e = \underline{\underline{3.2}}$ (rounded to the nearest 0.1)

<u>10 yr event</u>

2700 cfs $\quad H \cong 0.175$

$\frac{H}{P} = \frac{0.175'}{0.958'} = \underline{0.183}$

$C_e = \underline{\underline{3.2}}$

<u>20 yr event</u>

3400 cfs $\quad H \cong 0.203$

$\frac{H}{P} = \frac{0.203}{0.958} = \underline{0.212}$

$C_e = \underline{\underline{3.2}}$

<u>100 yr event</u>

5000 cfs $\quad H \cong .263$

$\frac{H}{P} = \frac{.263}{.958} = \underline{0.275}$

$C_e = \underline{\underline{3.2}}$

Figure A2. Weir coefficient calculations

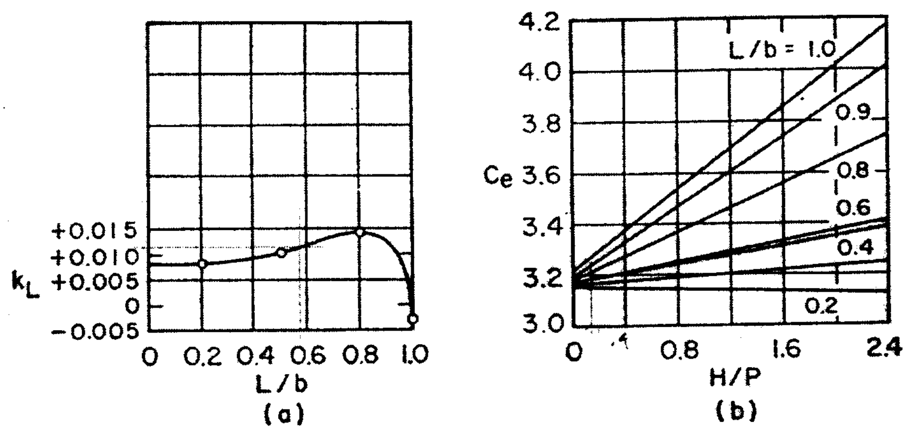


FIG. 5-3. Coefficients, horizontal sharp-crested weirs with end contractions.

NATURAL CHANNEL

Figure A3. Graphs used in weir coefficient calculations

BYPASS CHANNEL

$\frac{L}{b} = \frac{2.5'}{5.25'} = \underline{0.476}$ P= 0.958'

$K_L = ^{+}0.010$, Fig 5-3

$L_e = 2.5 + 0.010 = 2.51$ used 2.522

1.2 yr event

0 → no flow

10 yr event

4400 cfs , $H \cong 0.231$

$\frac{H}{P} = \frac{0.231}{0.958} = \underline{0.241}$

$C_e = 3.2$

20 yr event

5600 cfs , $H \cong .271$

$\frac{H}{P} = \frac{.271}{.958} = \underline{0.283}$

$C_e = 3.2$

100 yr event

9600 cfs , $H \cong .388$

$\frac{H}{P} = \frac{0.388}{0.958} = \underline{0.405}$

$C_e = 3.2$

Figure A4. Weir coefficient calculations

5–14 HANDBOOK OF HYDRAULICS

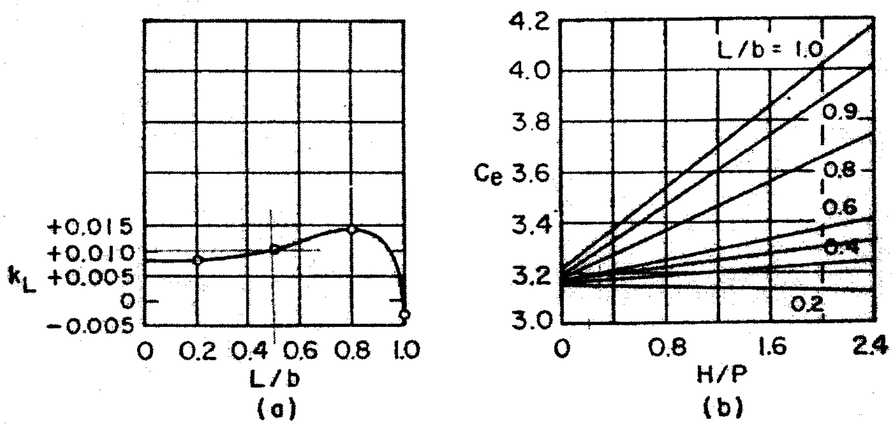


Fig. 5-3. Coefficients, horizontal sharp-crested weirs with end contractions.

BYPASS CHANNEL

Figure A5. Graphs used in weir coefficient calculations

Geometry

Weir Length $L_o := 30 \cdot in$

Weir Height $P := 0.958 \cdot ft$

$i := 0..6$

Head $h_0 := 0.118 \cdot ft$ $h_1 := 0.175 \cdot ft$ $h_2 := 0.203 \cdot ft$ $h_3 := 0.231 \cdot ft$

$h_4 := 0.263 \cdot ft$ $h_5 := 0.271 \cdot ft$ $h_6 := 0.388 \cdot ft$

Effective Weir Length Equation 13 page 215 from Rouse Engr Hyd

$n := 2$

$$L_i := L_o - \frac{n}{10} \cdot h_i$$

Discharge Coefficient Equation 12 page 214 from Rouse Engr Hyd

$$C_{d_i} := 0.605 + 0.08 \cdot \frac{h_i}{P} + \frac{1 \cdot ft}{305 \cdot h_i}$$

$$C_d = \begin{pmatrix} 0.643 \\ 0.638 \\ 0.638 \\ 0.638 \\ 0.639 \\ 0.640 \\ 0.646 \end{pmatrix}$$

Discharge Equation 11 page 213 from Rouse Engr Hyd

$$Q_i := C_{d_i} \cdot \frac{2}{3} \sqrt{2 \cdot g} \cdot L_i \cdot \left(h_i\right)^{\frac{3}{2}} \cdot 30^{\frac{5}{2}}$$

$$Q = \begin{pmatrix} 1701 \\ 3037 \\ 3784 \\ 4585 \\ 5564 \\ 5819 \\ 9968 \end{pmatrix} \frac{ft^3}{sec}$$

WES Discharge Equation from King's Handbook

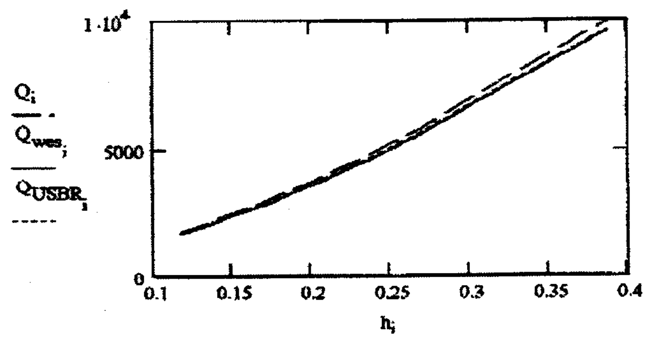
$$Q_{wes_i} := 3.2 \cdot ft^{.5} \cdot sec^{-1} \cdot \left(\left(L_o + 0.012 \cdot ft\right)\right) \cdot \left(h_i\right)^{\frac{3}{2}} \cdot 30^{\frac{5}{2}}$$

$$Q_{wes} = \begin{pmatrix} 1606 \\ 2901 \\ 3624 \\ 4399 \\ 5344 \\ 5590 \\ 9577 \end{pmatrix} \frac{ft^3}{sec}$$

Figure A6. Upper Guadalupe weir calibration

USBR Water Measurement Manual Equation 1 page 20

$$Q_{USBR_i} := 3.33 \cdot \left(h_i\right)^{\frac{3}{2}} \cdot \left(L_o - 0.2 \cdot h_i\right) \cdot 30^{\frac{5}{2}} \cdot ft^{-5} \cdot sec^{-1} \qquad Q_{USBR} = \begin{pmatrix} 1648 \\ 2962 \\ 3693 \\ 4472 \\ 5419 \\ 5664 \\ 9610 \end{pmatrix} \frac{ft^3}{sec}$$



Percent Differences

$$PCT_{USBR_i} := \frac{\left(Q_{USBR_i} - Q_{wes_i}\right)}{Q_{wes_i}} \cdot 100 \qquad PCT_{Rouse_i} := \frac{\left(Q_i - Q_{wes_i}\right)}{Q_{wes_i}} \cdot 100$$

$$PCT_{USBR} = \begin{pmatrix} 2.6 \\ 2.1 \\ 1.9 \\ 1.7 \\ 1.4 \\ 1.3 \\ 0.4 \end{pmatrix} \qquad PCT_{Rouse} = \begin{pmatrix} 5.9 \\ 4.7 \\ 4.4 \\ 4.2 \\ 4.1 \\ 4.1 \\ 4.1 \end{pmatrix}$$

Conclusions

The USBR Water Measurement Manual gives discharges that are between 0.4 and 2.6 percent higher than King's Handbook. The equations of Rouse give values of discharge that are between 4.1 and 5.9 percent higher than King's Handbook. The variance between the methods is within experimental error. More precision can be obtained only by a detailed in place calibration of the weirs used by WES. However, I believe that the existing values that were used by WES are sufficiently accurate for the purposes of this model study.

Figure A7. Upper Guadalupe weir calibration

Appendix B
Water-Surface Profiles, Design Roughness

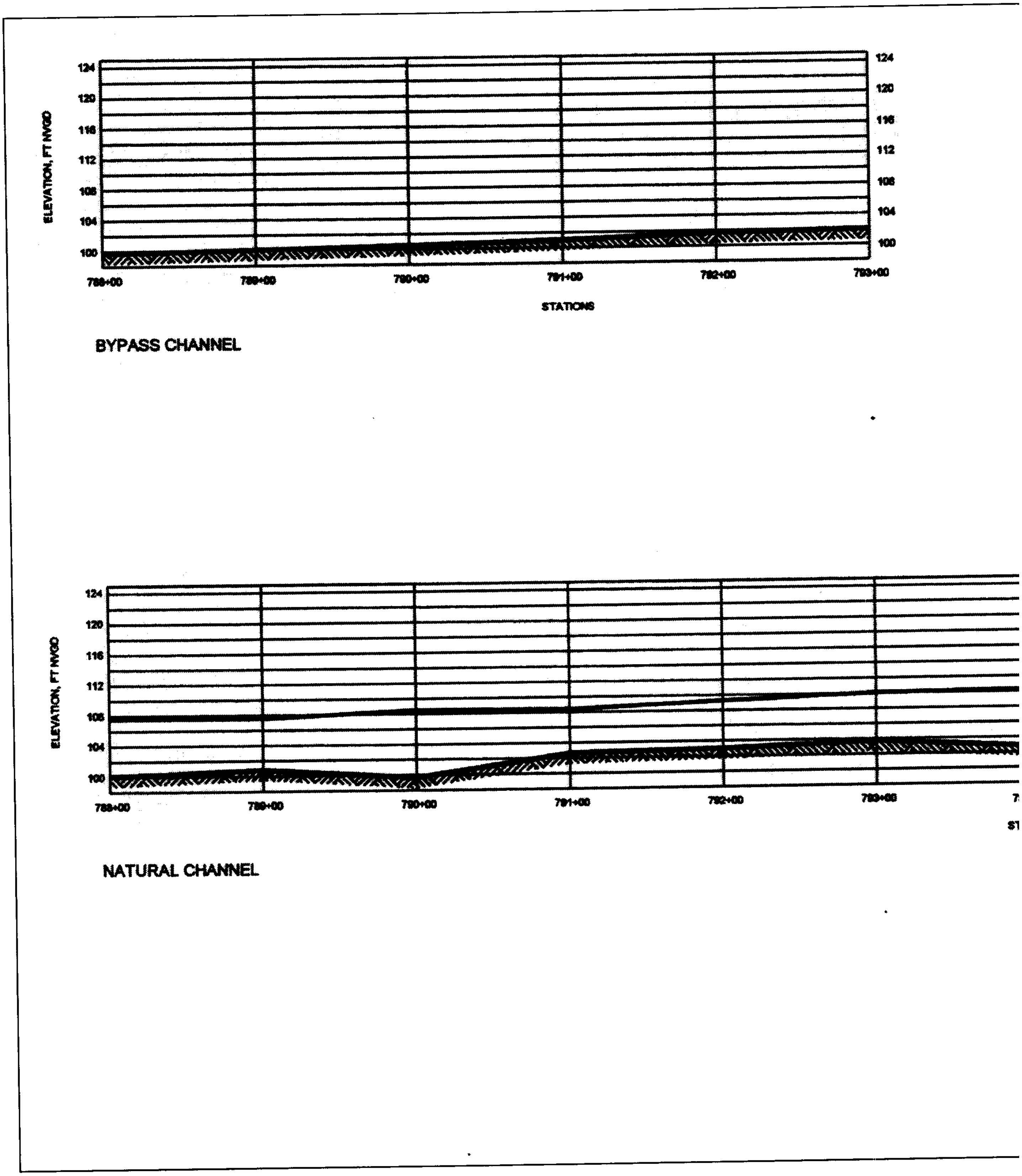
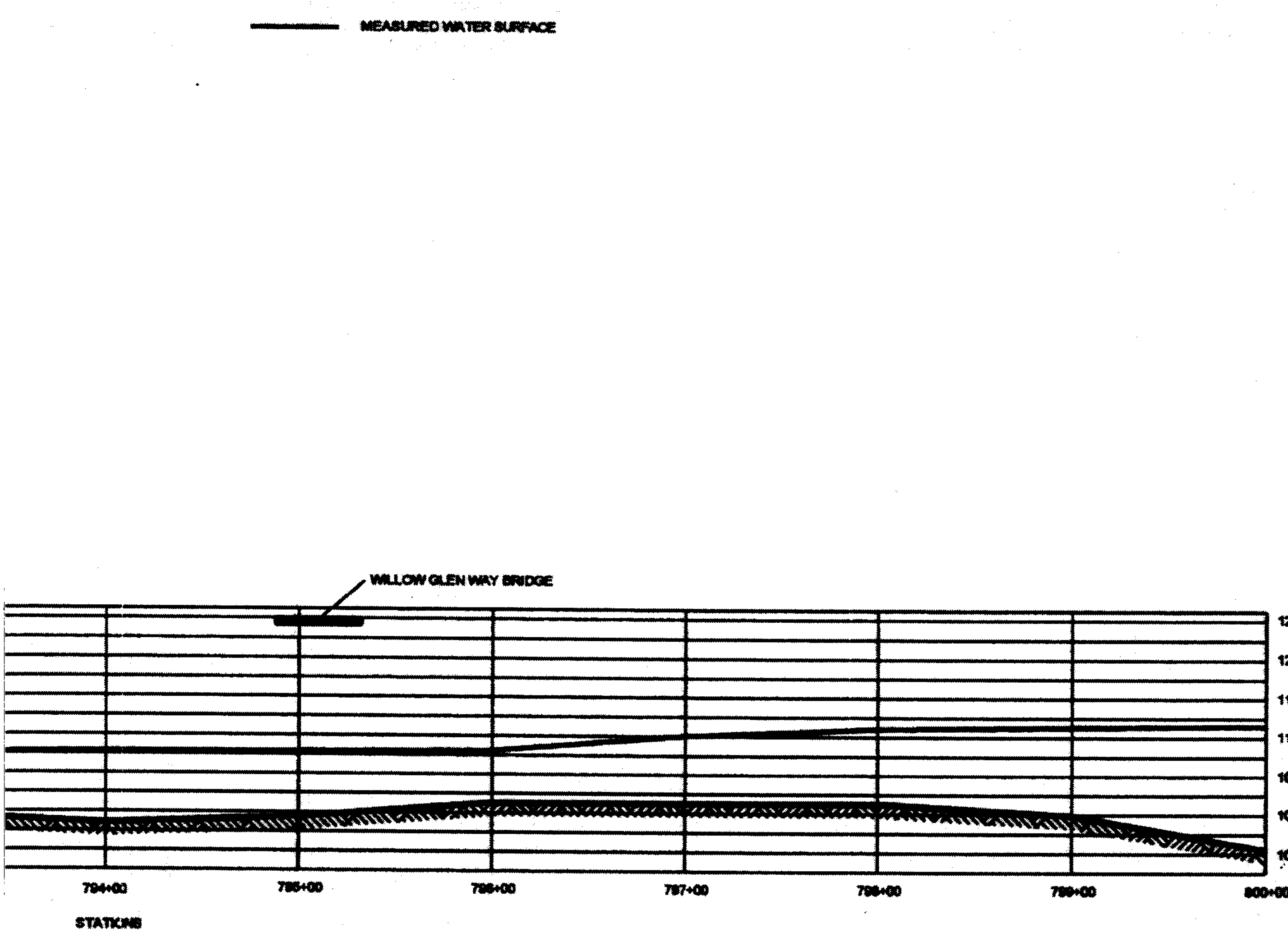
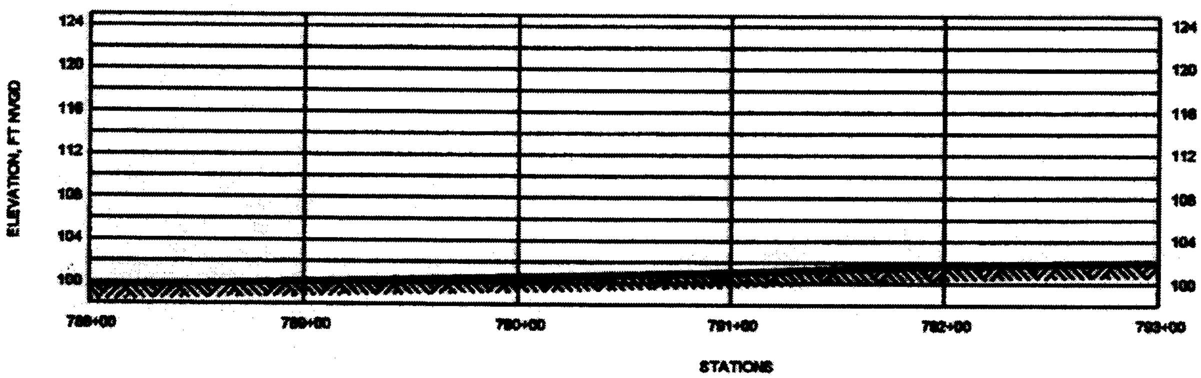

124
120
116
112
108
104
100
ELEVATION, FT NVGD
788+00
789+00
790+00
791+00
792+00
793+00
STATIONS
BYPASS CHANNEL
NATURAL CHANNEL

Figure B1.

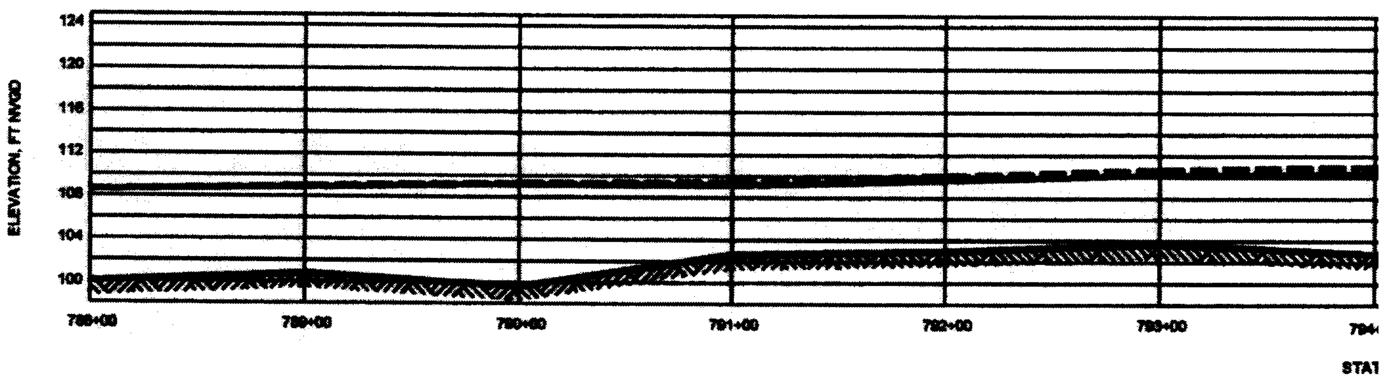


WATER-SURFACE PROFILES
DESIGN ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 0 CFS - NATURAL CHANNEL 1500 CFS)

2

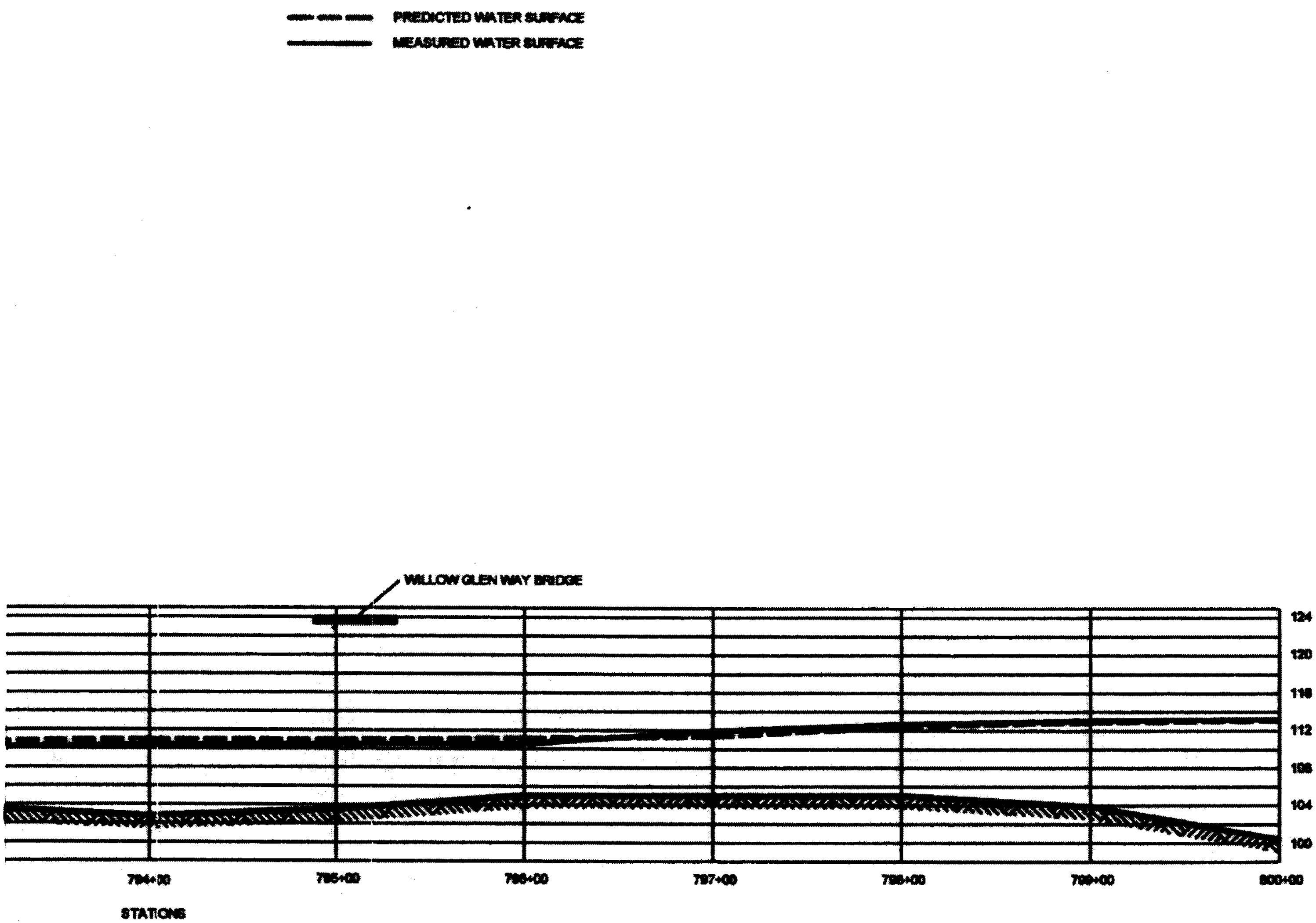


BYPASS CHANNEL



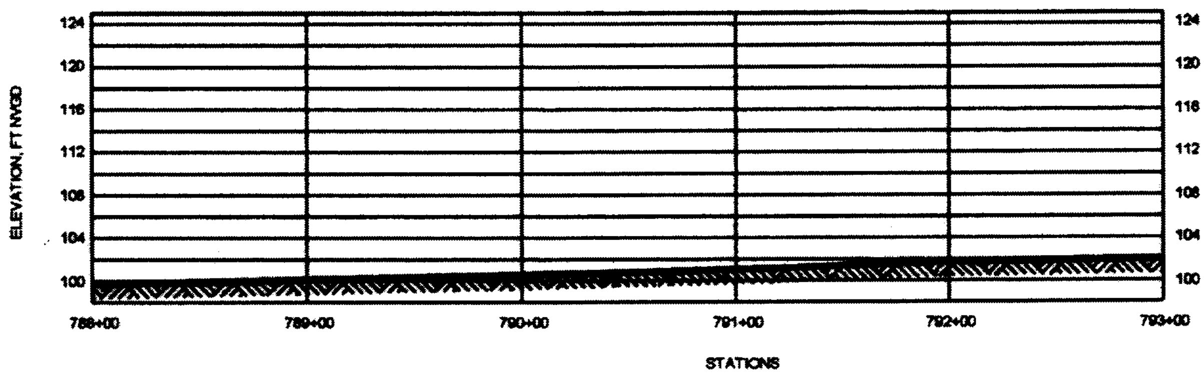
NATURAL CHANNEL

Figure B2.

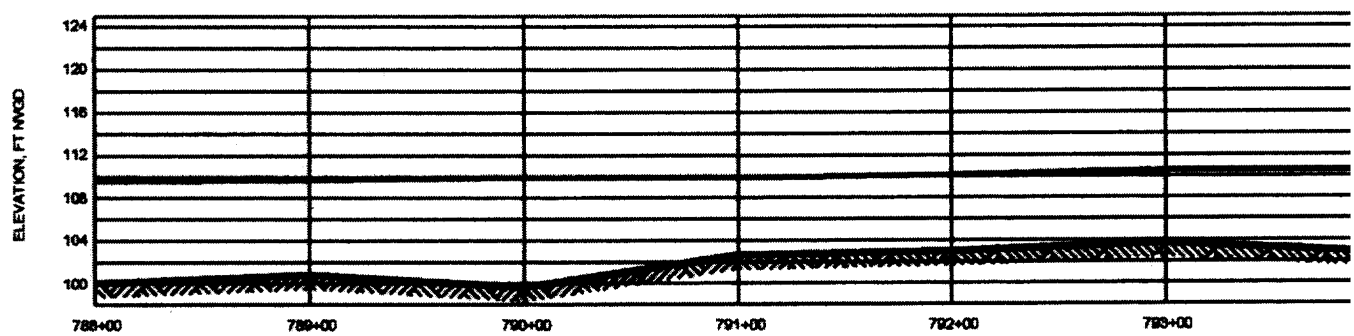


WATER-SURFACE PROFILES
DESIGN ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER
(BYPASS CHANNEL 0 CFS - NATURAL CHANNEL 1500 CFS)

2

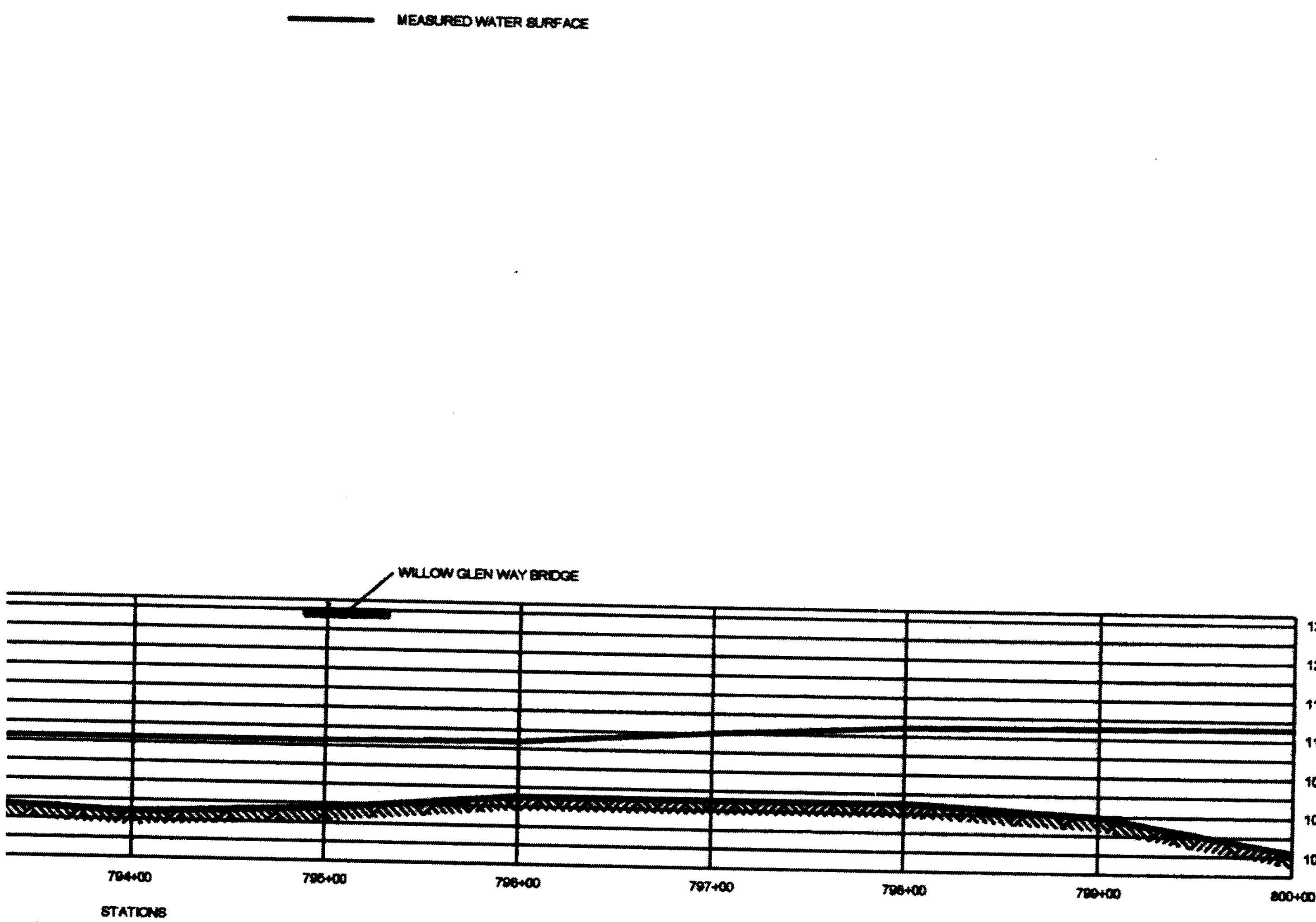


BYPASS CHANNEL



NATURAL CHANNEL

Figure B3.



WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 0 CFS - NATURAL CHANNEL 1500 CFS)

2

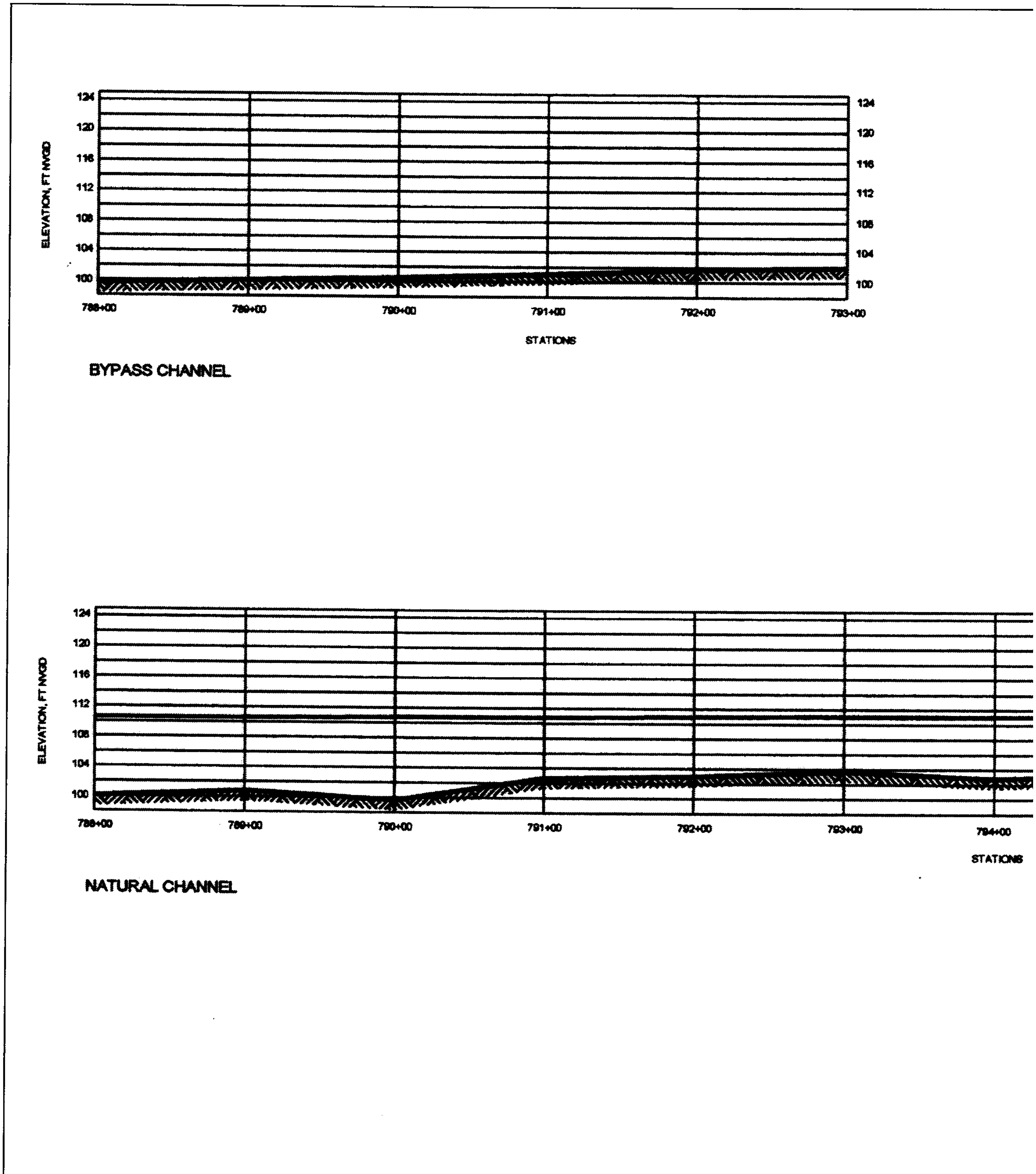
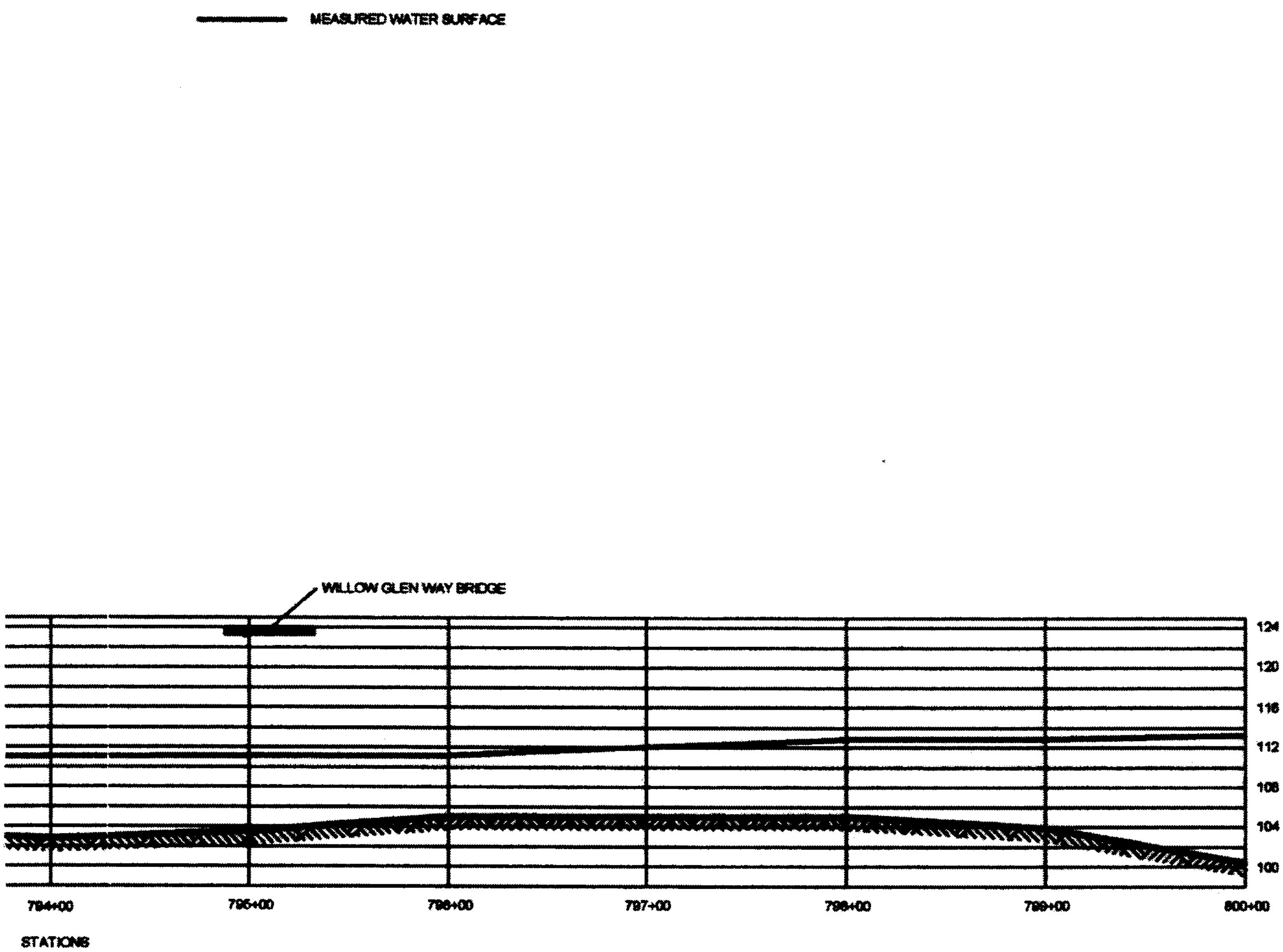
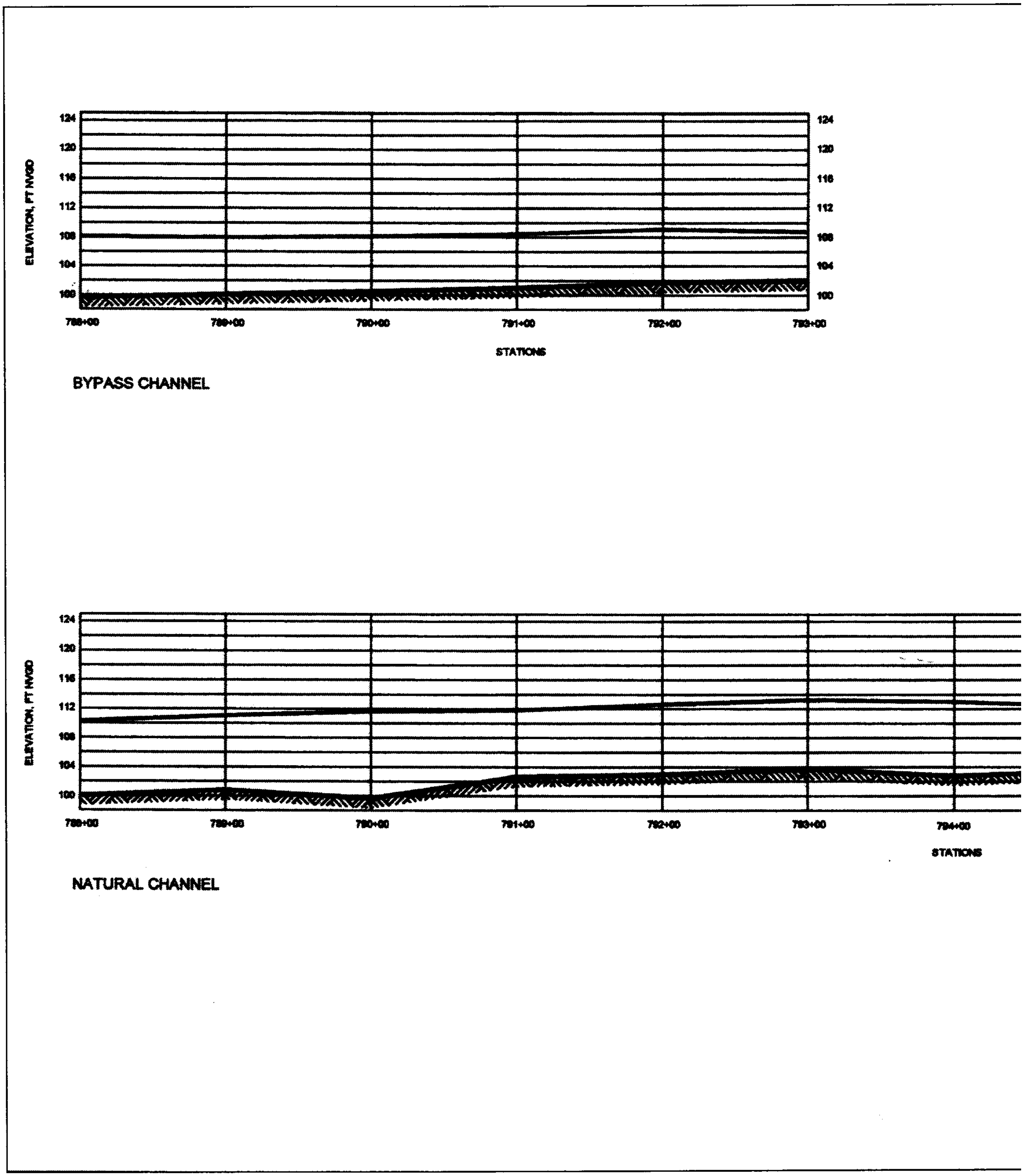

ELEVATION, FT NVGD
124
120
116
112
108
104
100
788+00
789+00
790+00
791+00
792+00
793+00
STATIONS
BYPASS CHANNEL
ELEVATION, FT NVGD
124
120
116
112
108
104
100
788+00
789+00
790+00
791+00
792+00
793+00
794+00
STATIONS
NATURAL CHANNEL

Figure B4.



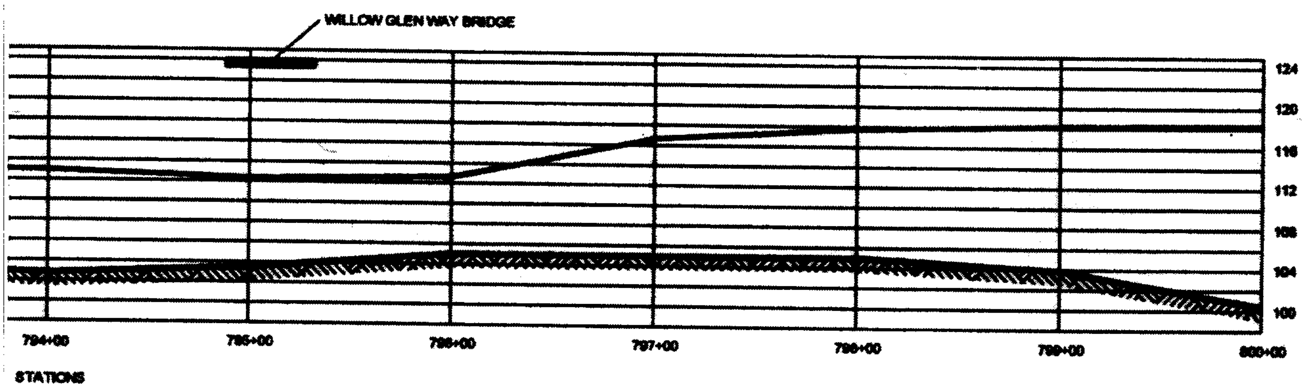
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER +2.0'
(BYPASS CHANNEL 200 CFS - NATURAL CHANNEL 1300 CFS)



ELEVATION, FT NVGD
124
120
116
112
108
104
100
STATIONS
BYPASS CHANNEL
ELEVATION, FT NVGD
STATIONS
NATURAL CHANNEL

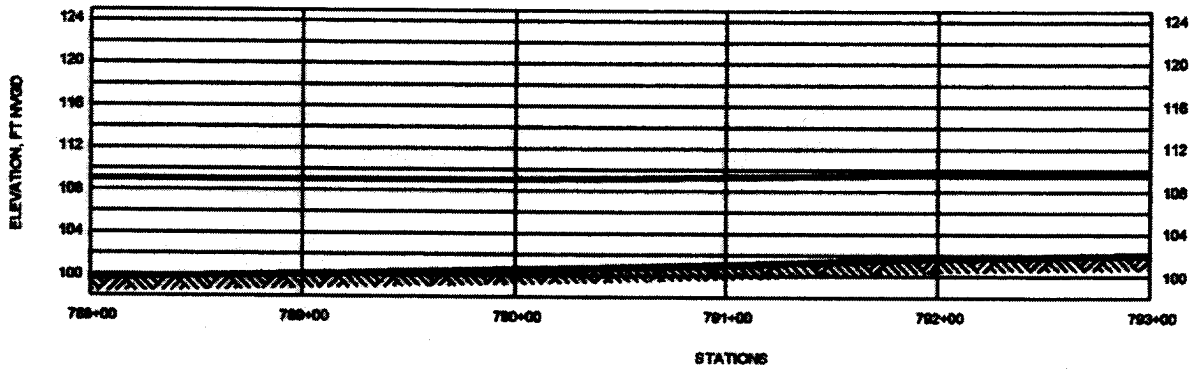
Figure B5.



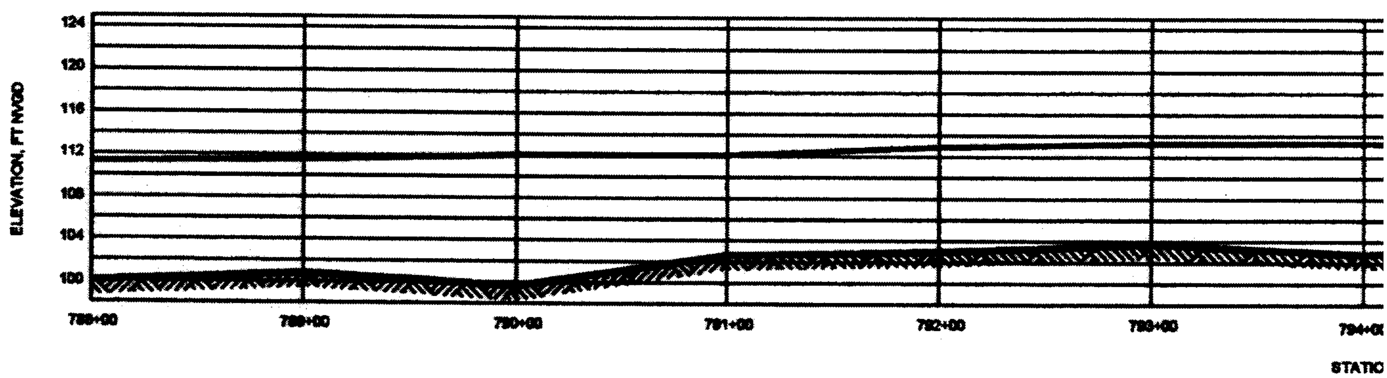
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 7100 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 3650 CFS - NATURAL CHANNEL 3450 CFS)

2

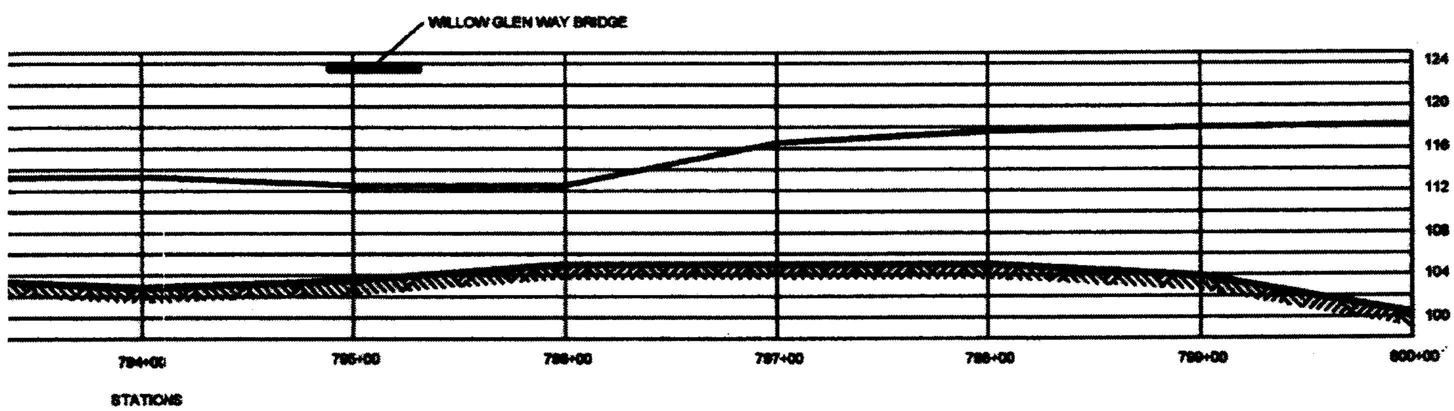


BYPASS CHANNEL



NATURAL CHANNEL

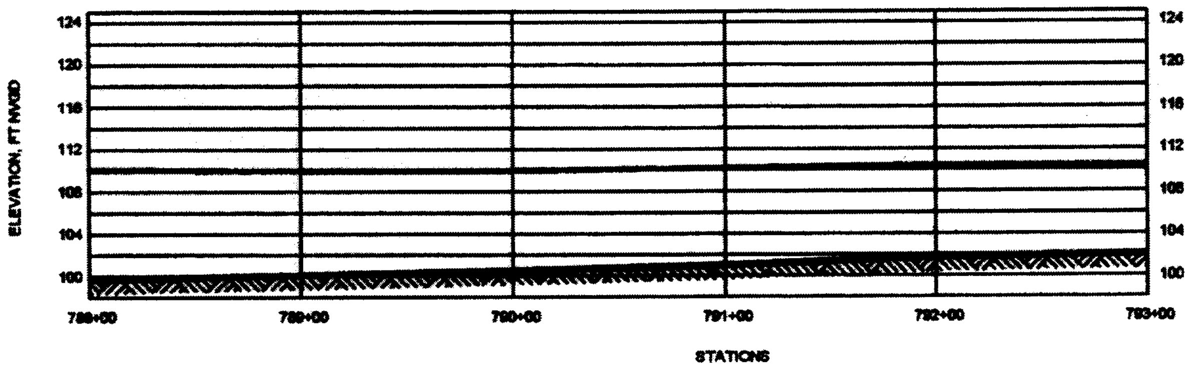
Figure B6.



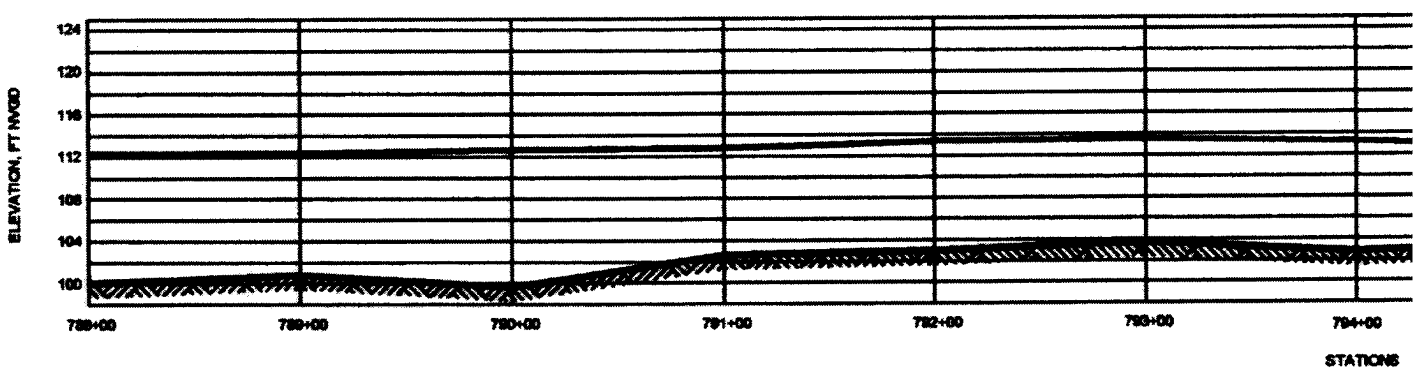
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 7100 CFS DESIGN TAILWATER
(BYPASS CHANNEL 3735 CFS - NATURAL CHANNEL 3365 CFS)

2

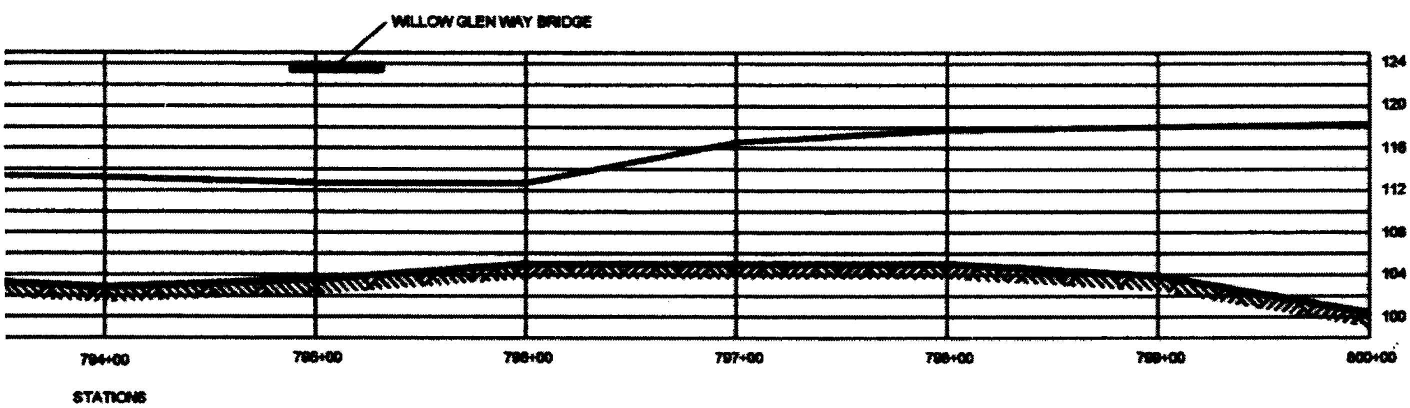


BYPASS CHANNEL



NATURAL CHANNEL

Figure B7.



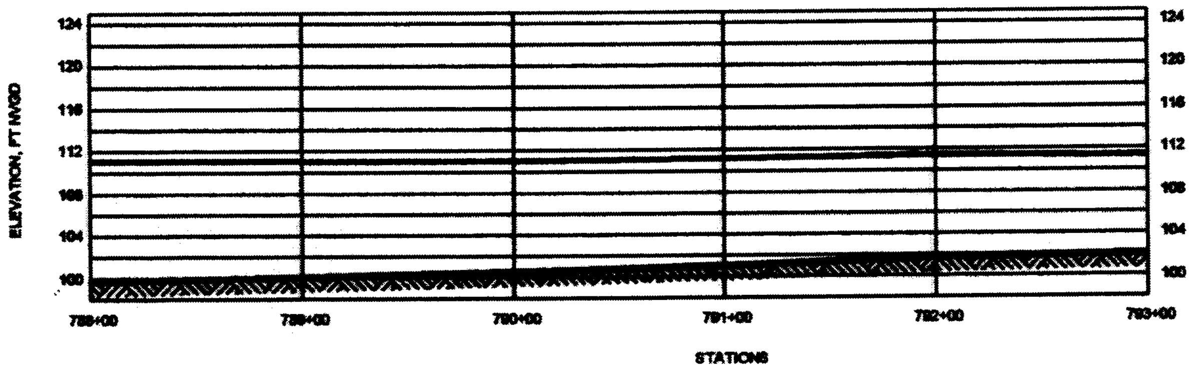
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN

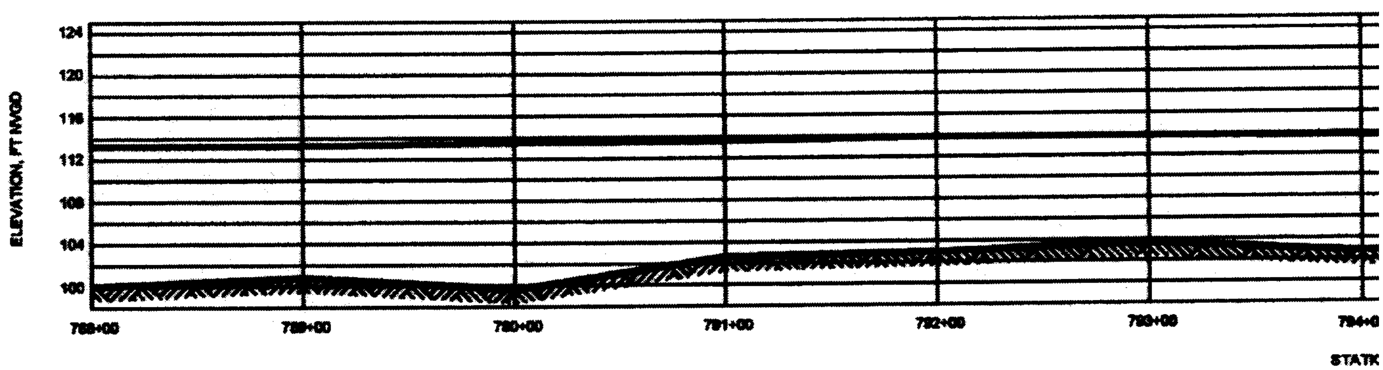
DISCHARGE 7100 CFS DESIGN TAILWATER +1.0'

(BYPASS CHANNEL 3850 CFS - NATURAL CHANNEL 3250 CFS)

2

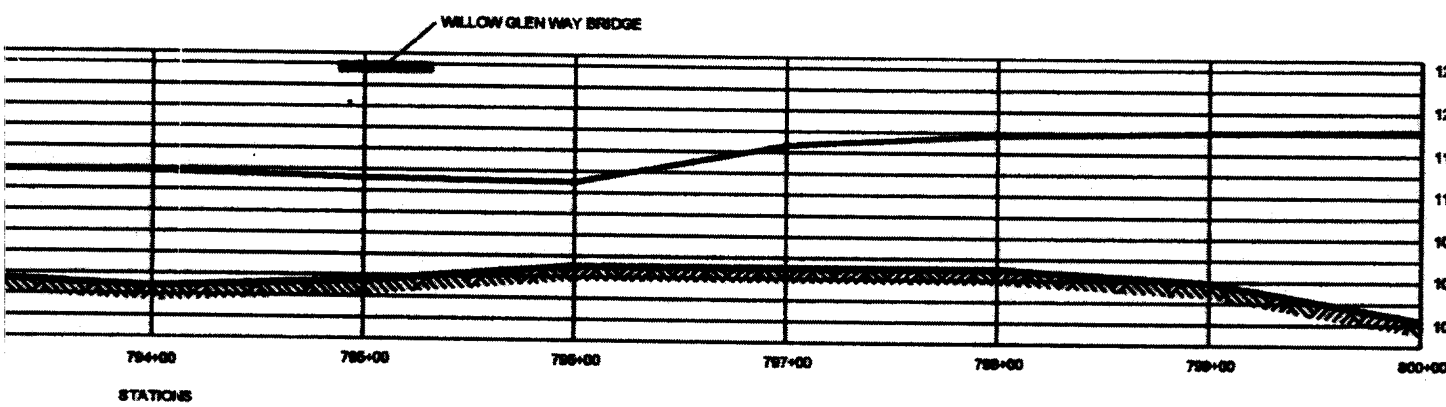


BYPASS CHANNEL



NATURAL CHANNEL

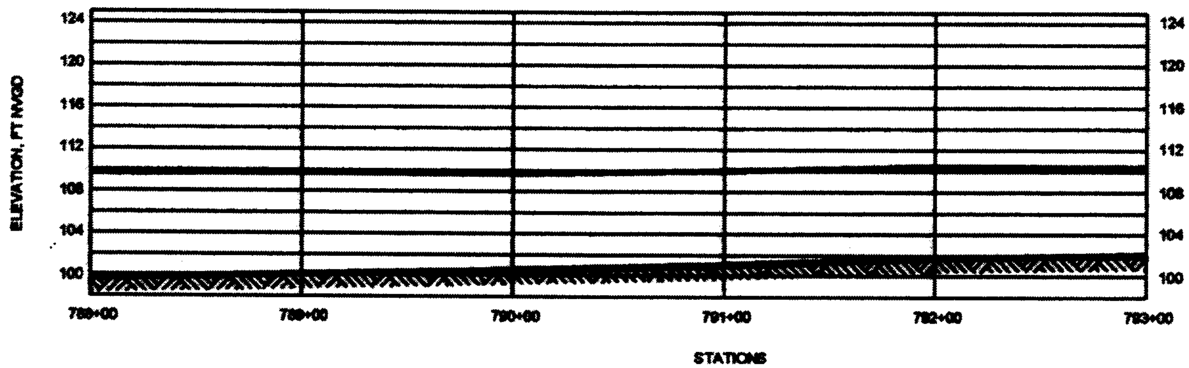
Figure B8.



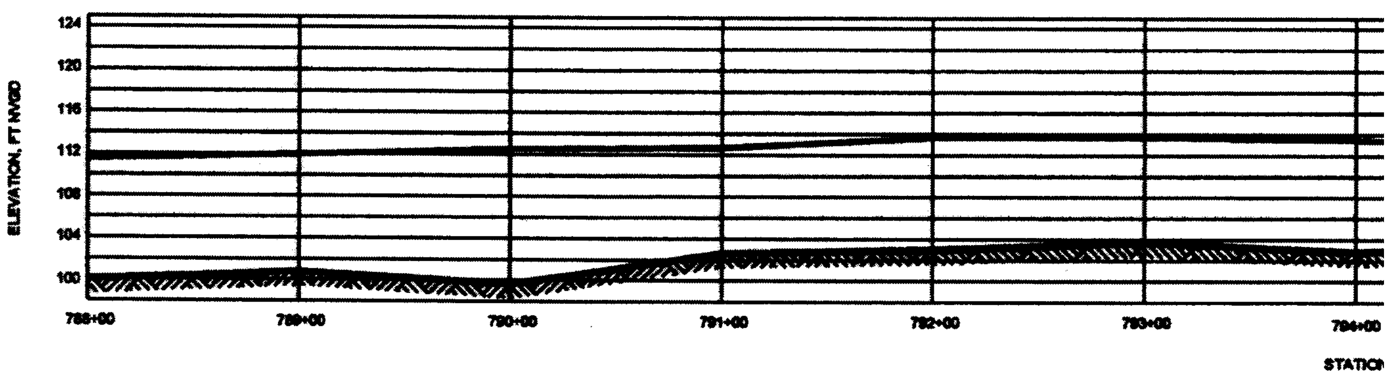
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 7100 CFS DESIGN TAILWATER +2.0'
(BYPASS CHANNEL 4350 CFS - NATURAL CHANNEL 2750 CFS)

2

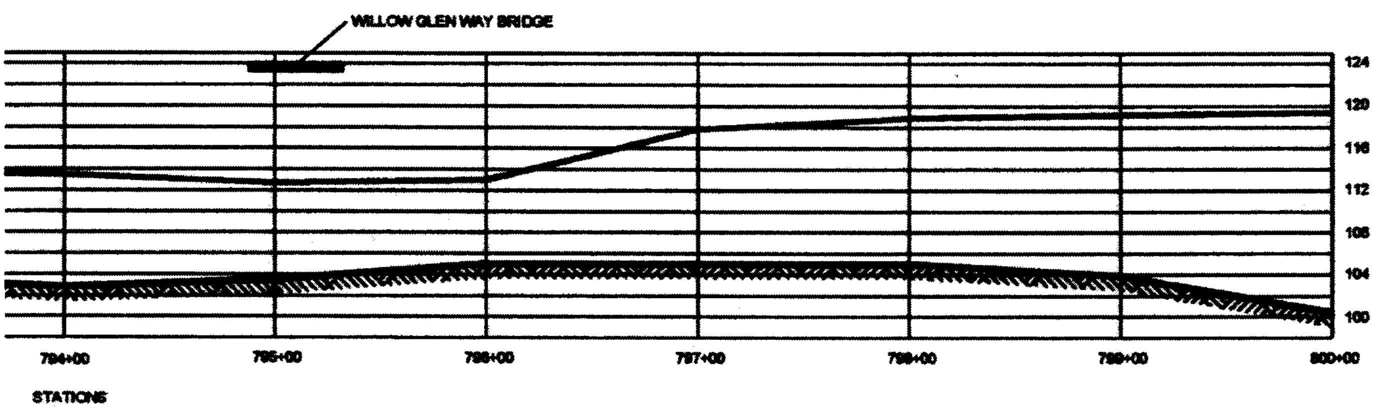


BYPASS CHANNEL



NATURAL CHANNEL

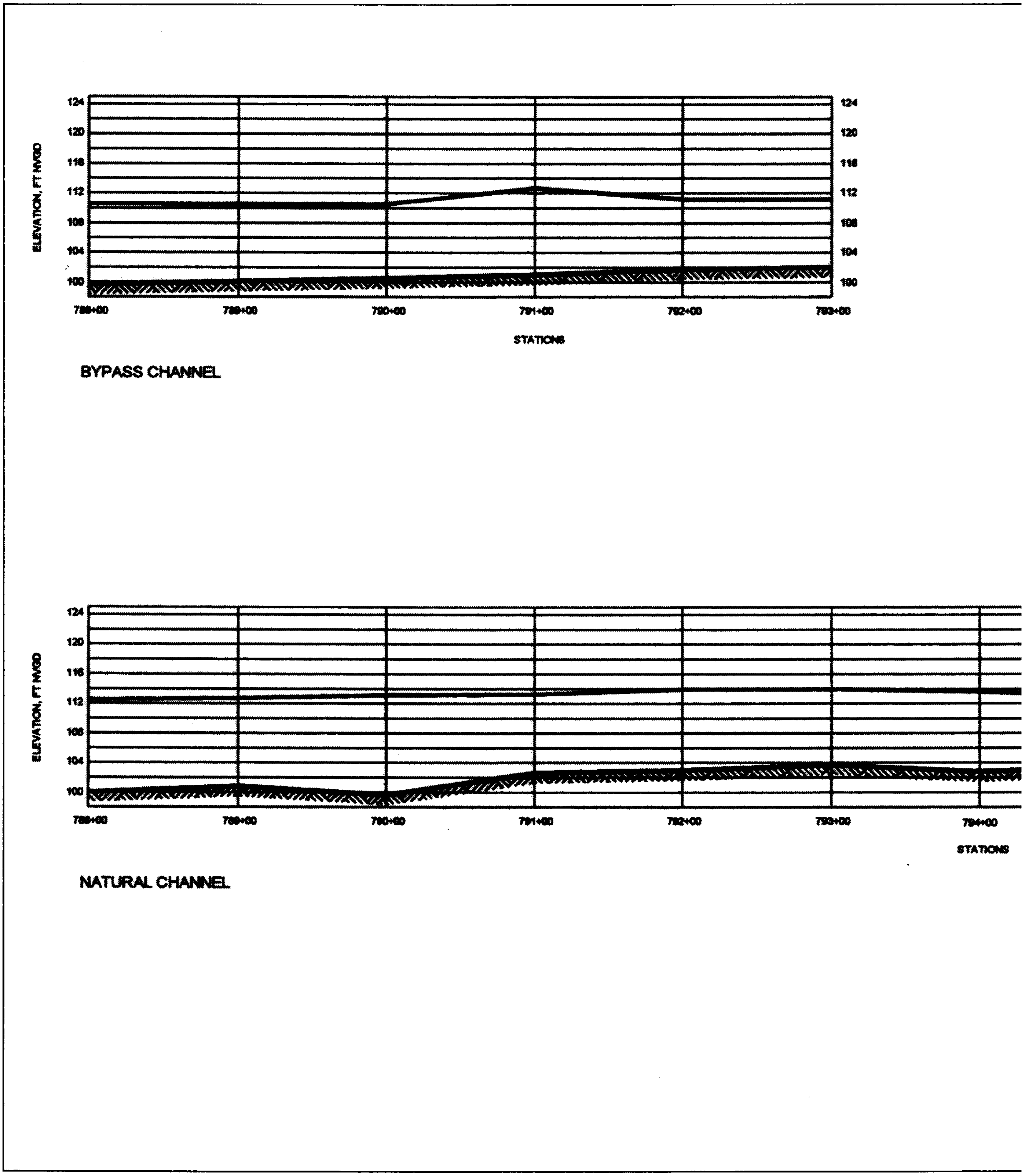
Figure B9.



WATER-SURFACE PROFILES
DESIGN ROUGHNESS

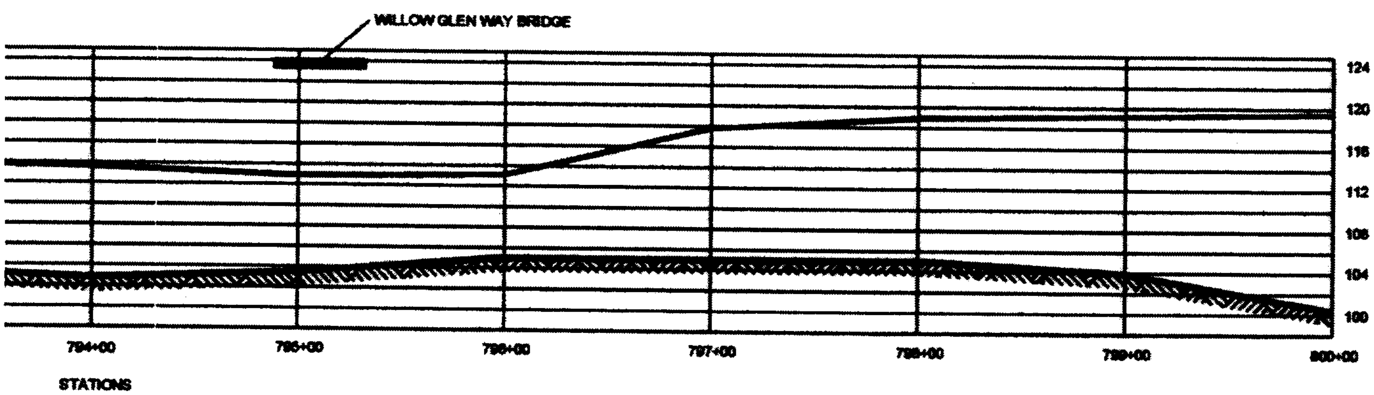
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 9000 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 5100 CFS - NATURAL CHANNEL 3900 CFS)

2



ELEVATION, FT NVGD
124
120
116
112
108
104
100
788+00
789+00
790+00
791+00
792+00
793+00
STATIONS
BYPASS CHANNEL
ELEVATION, FT NVGD
124
120
116
112
108
104
100
788+00
789+00
790+00
791+00
792+00
793+00
794+00
STATIONS
NATURAL CHANNEL

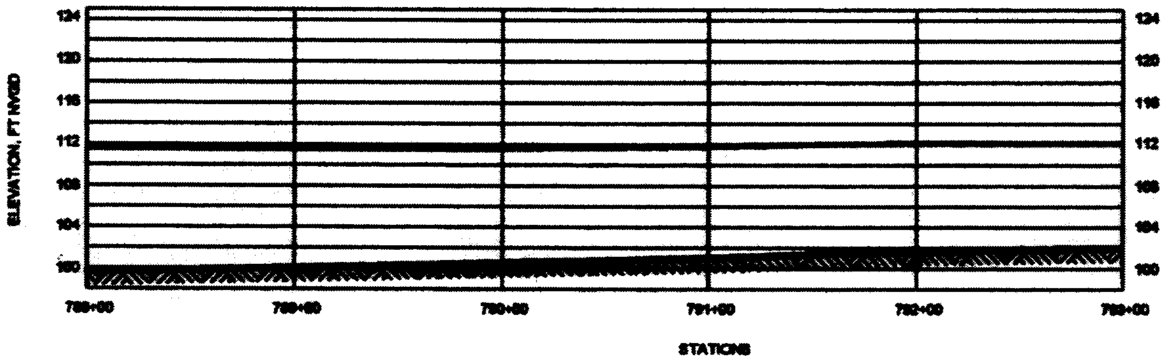
Figure B10.



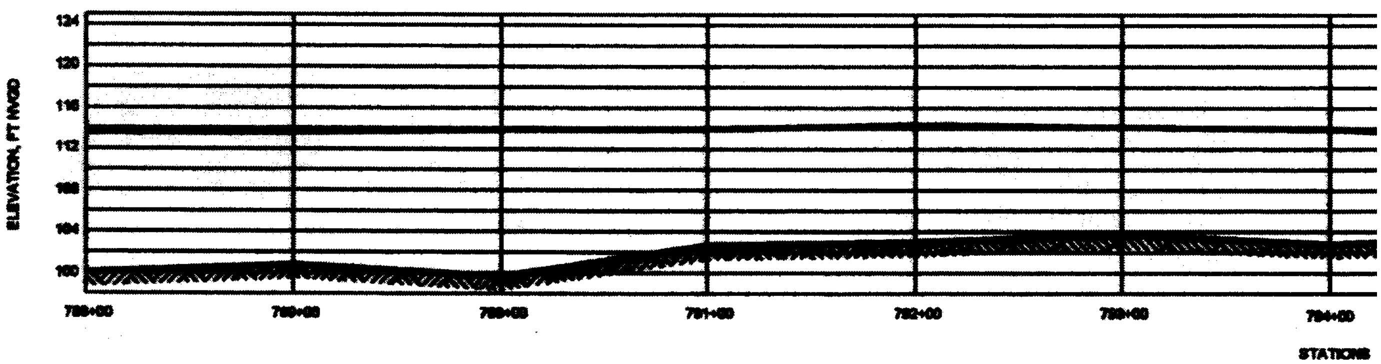
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 9000 CFS DESIGN TAILWATER
(BYPASS CHANNEL 3810 CFS - NATURAL CHANNEL 5190 CFS)

2

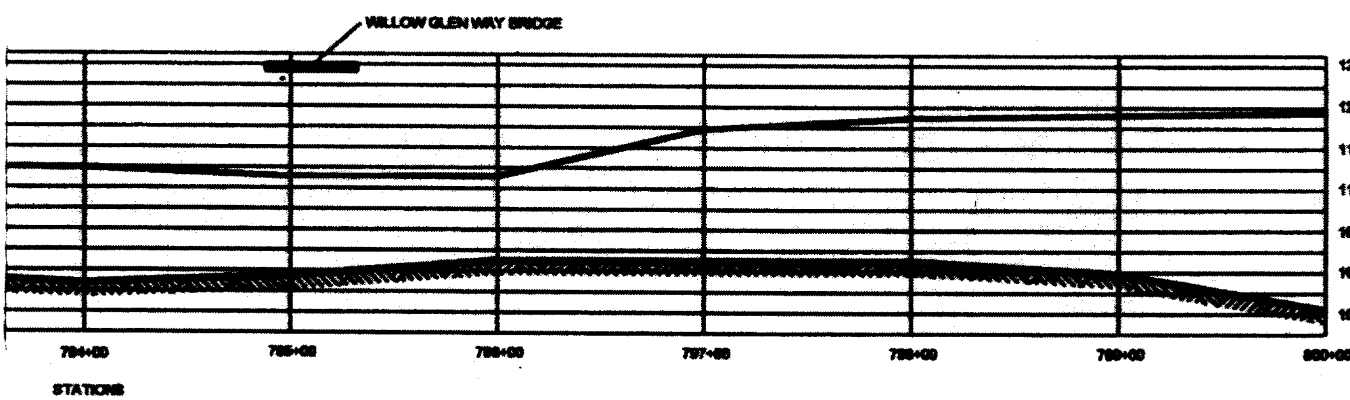


BYPASS CHANNEL



NATURAL CHANNEL

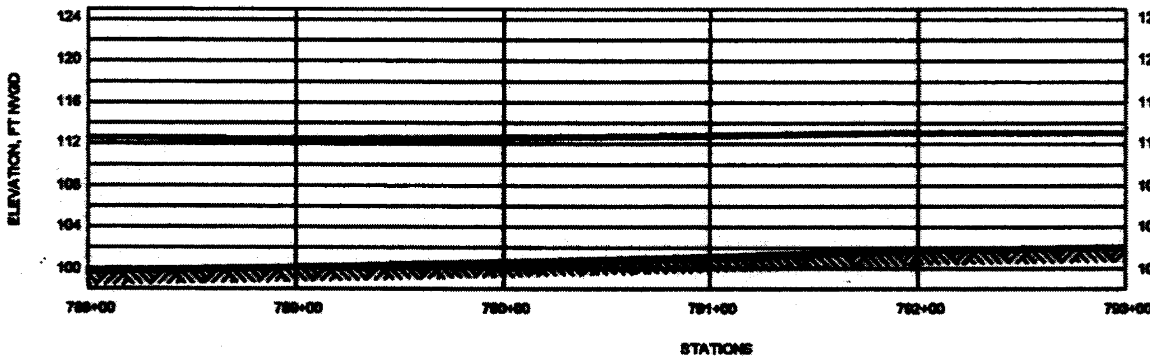
Figure B11.



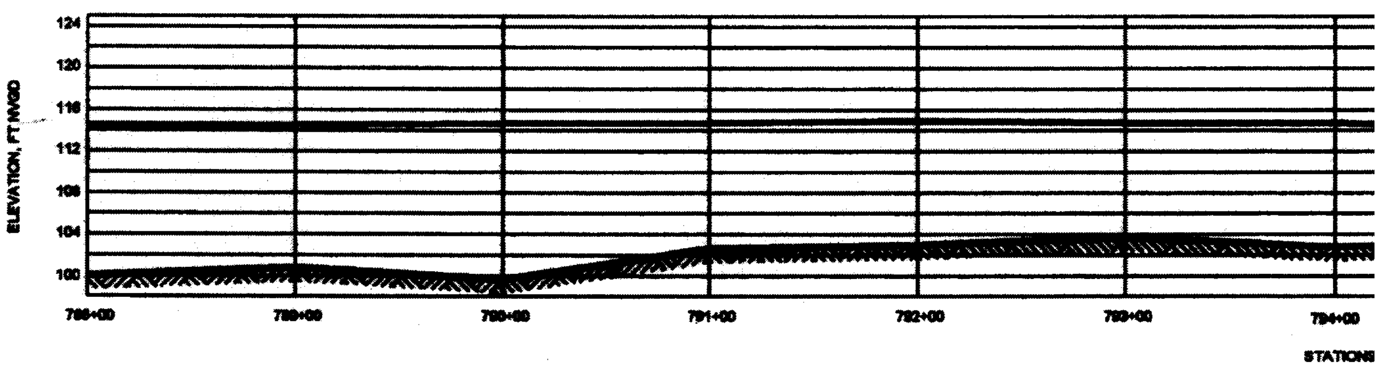
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 9000 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 5550 CFS - NATURAL CHANNEL 3450 CFS)

2

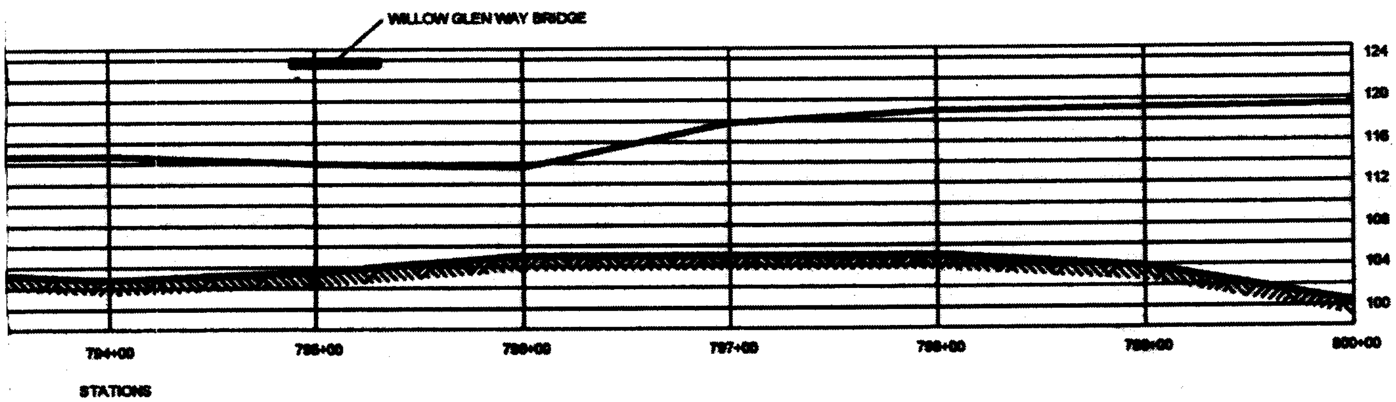


BYPASS CHANNEL



NATURAL CHANNEL

Figure B12.



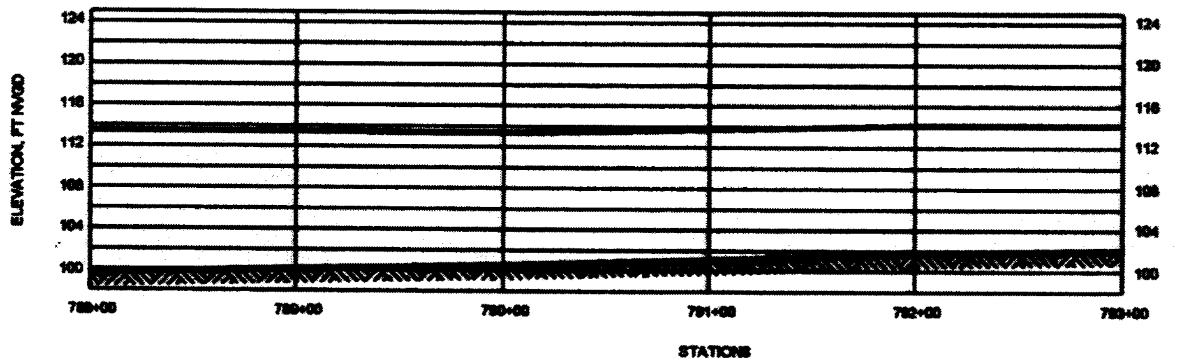
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN

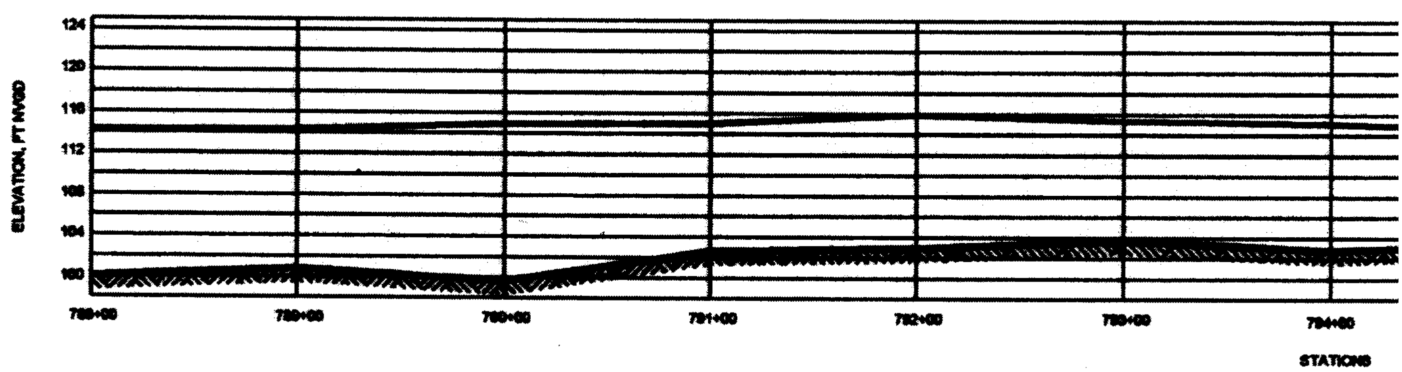
DISCHARGE 9000 CFS DESIGN TAILWATER +2.0'

(BYPASS CHANNEL 6050 CFS - NATURAL CHANNEL 2950 CFS)

2

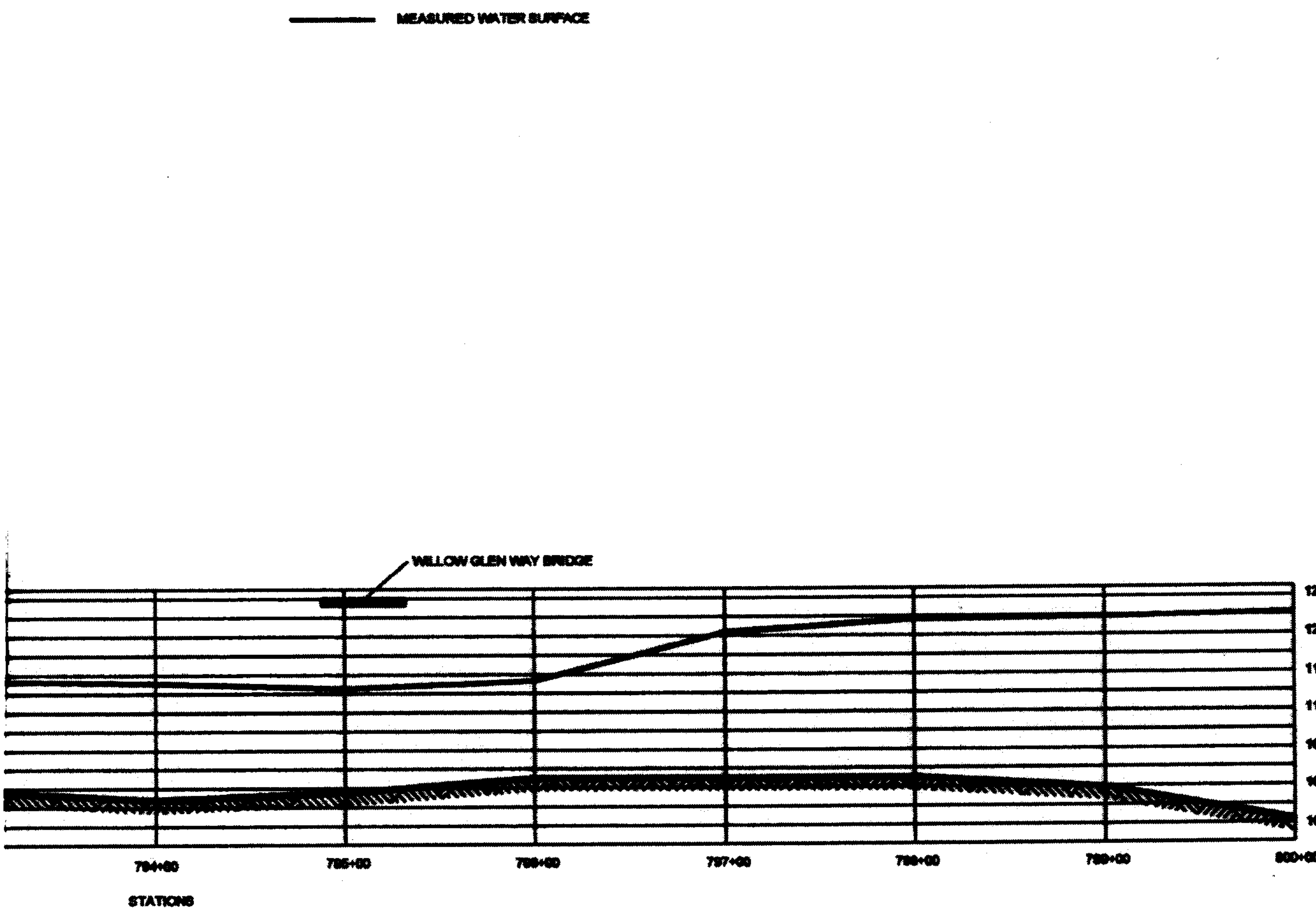


BYPASS CHANNEL



NATURAL CHANNEL

Figure B13.



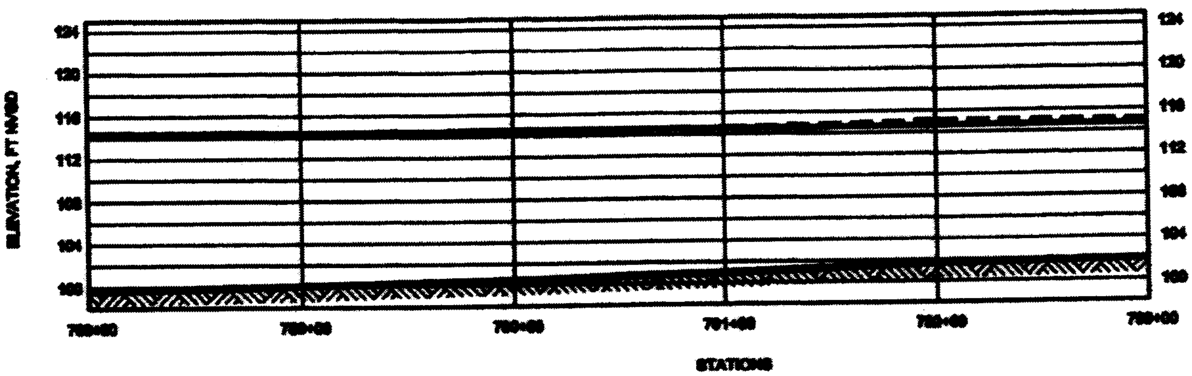
WATER-SURFACE PROFILES
DESIGN ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN

DISCHARGE 14600 CFS DESIGN TAILWATER -1.0'

(BYPASS CHANNEL 9600 CFS - NATURAL CHANNEL 5000 CFS)

2



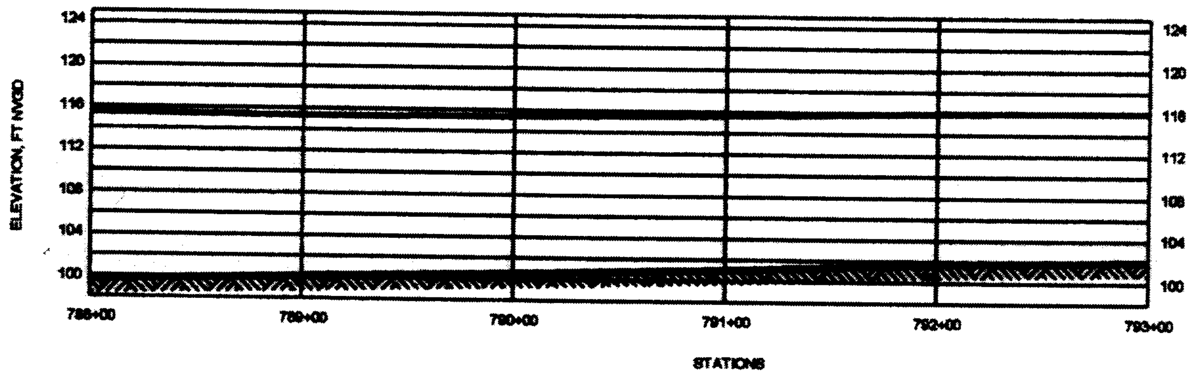
BYPASS CHANNEL

NATURAL CHANNEL

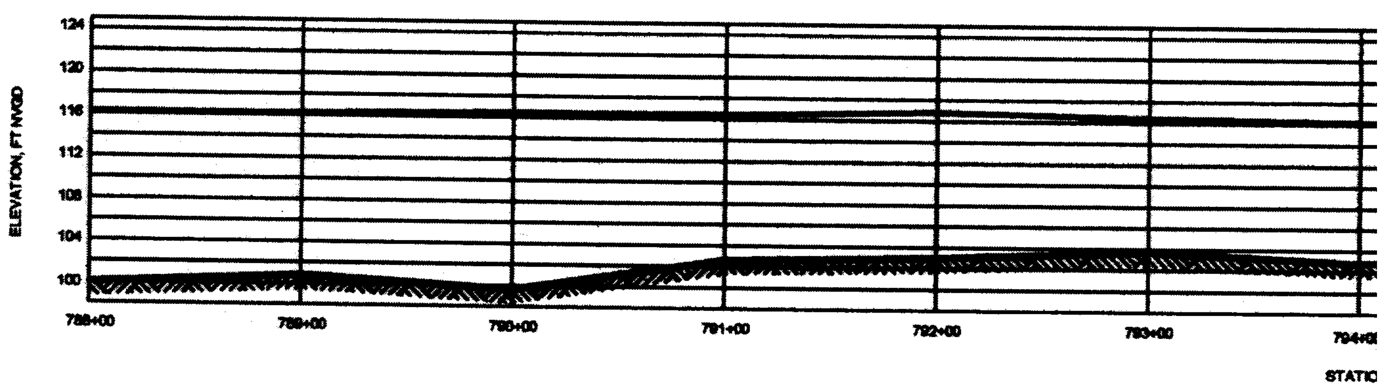
Figure B14.

WATER-SURFACE PROFILES
UPPER "n" VALUE
ALTERNATIVE 1 WEIR DESIGN
DISCHARGE 14600 CFS DESIGN TAILWATER
(BYPASS CHANNEL 9600 CFS - NATURAL CHANNEL 5000 CFS)

2

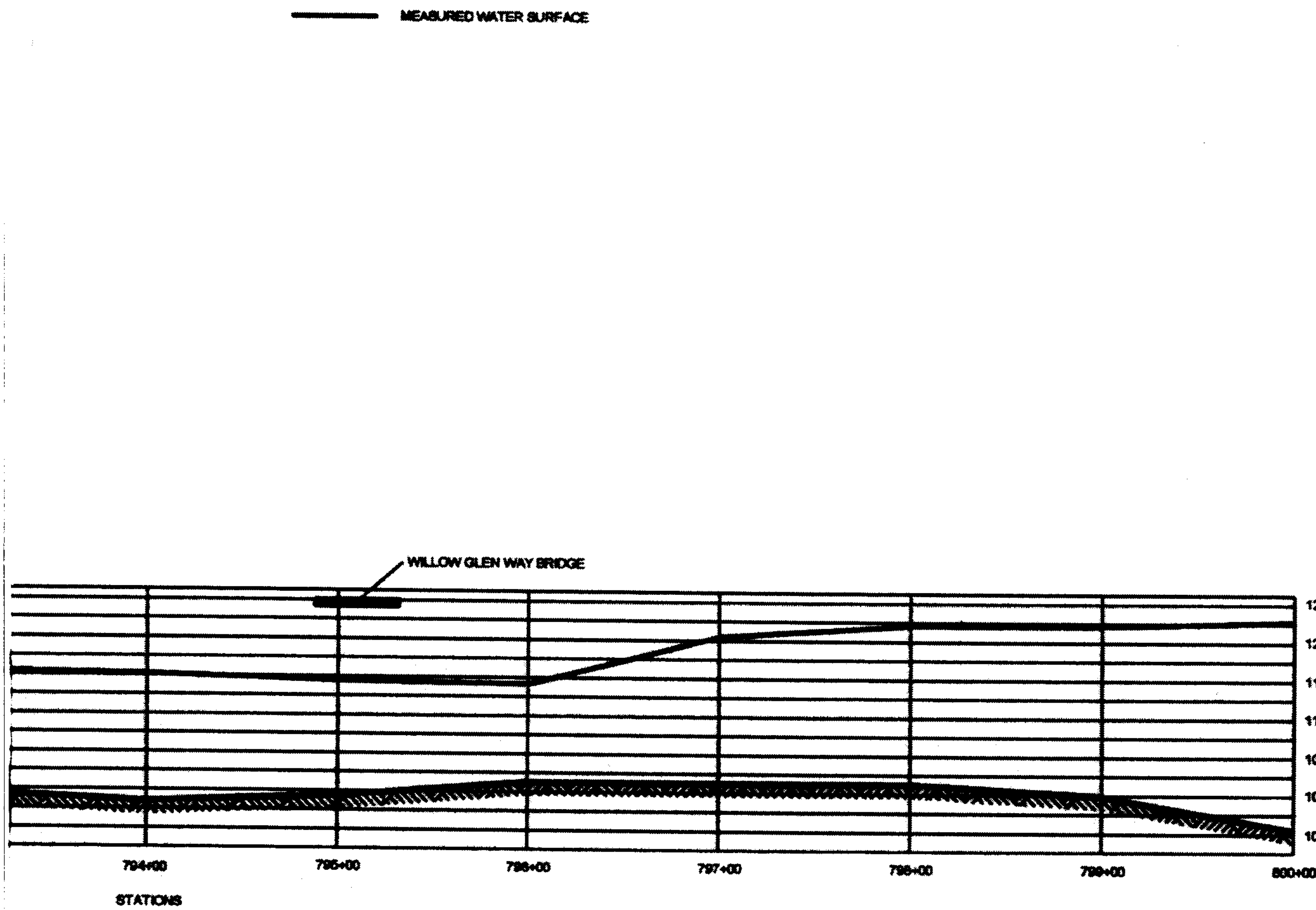


BYPASS CHANNEL



NATURAL CHANNEL

Figure B15.

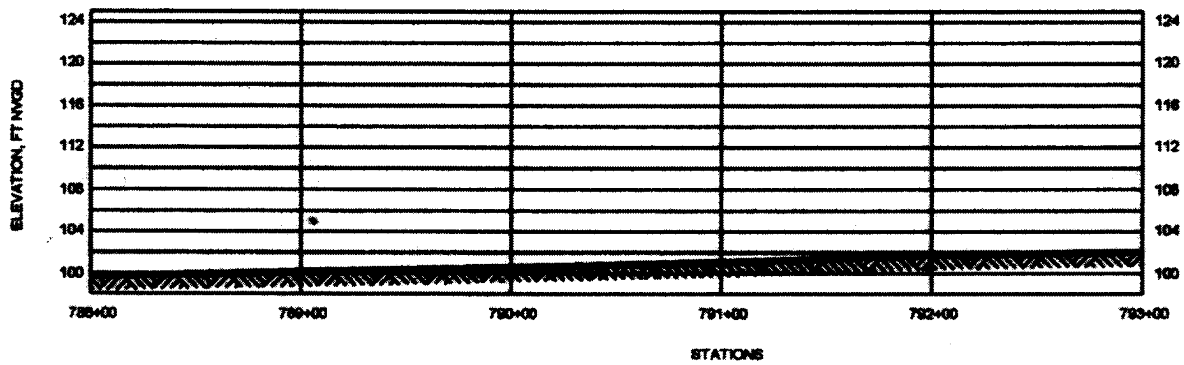


WATER-SURFACE PROFILES
DESIGN ROUGHNESS

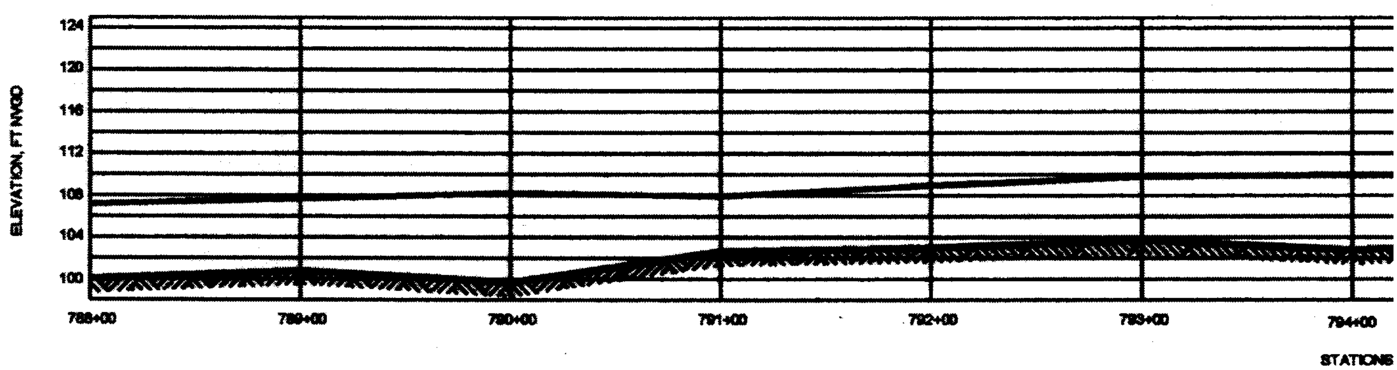
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 14600 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 9600 CFS - NATURAL CHANNEL 5000 CFS)

2

Appendix C
Water-Surface Profiles, New Channel Roughness

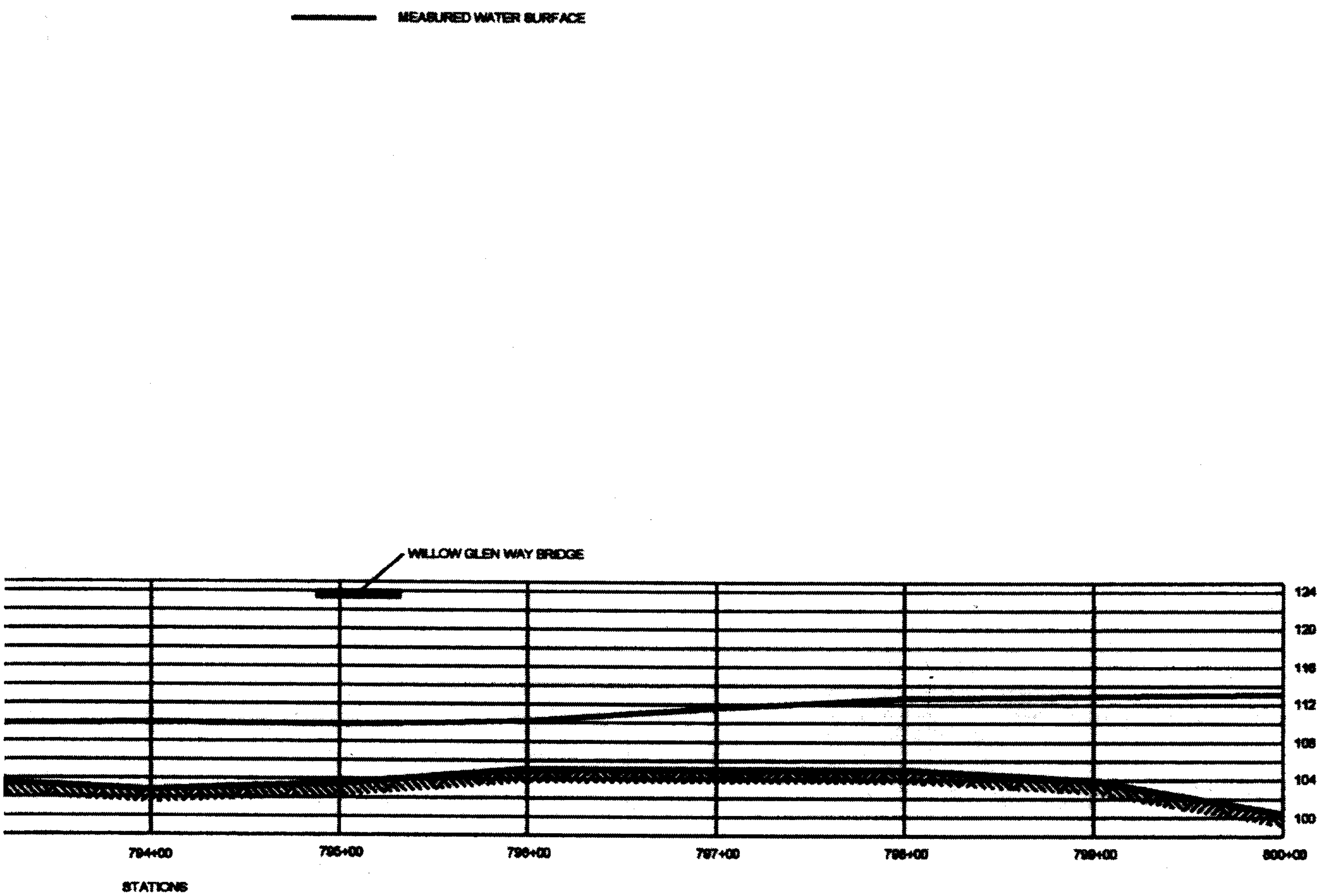


BYPASS CHANNEL



NATURAL CHANNEL

Figure C1.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 0 CFS - NATURAL CHANNEL 1500 CFS)

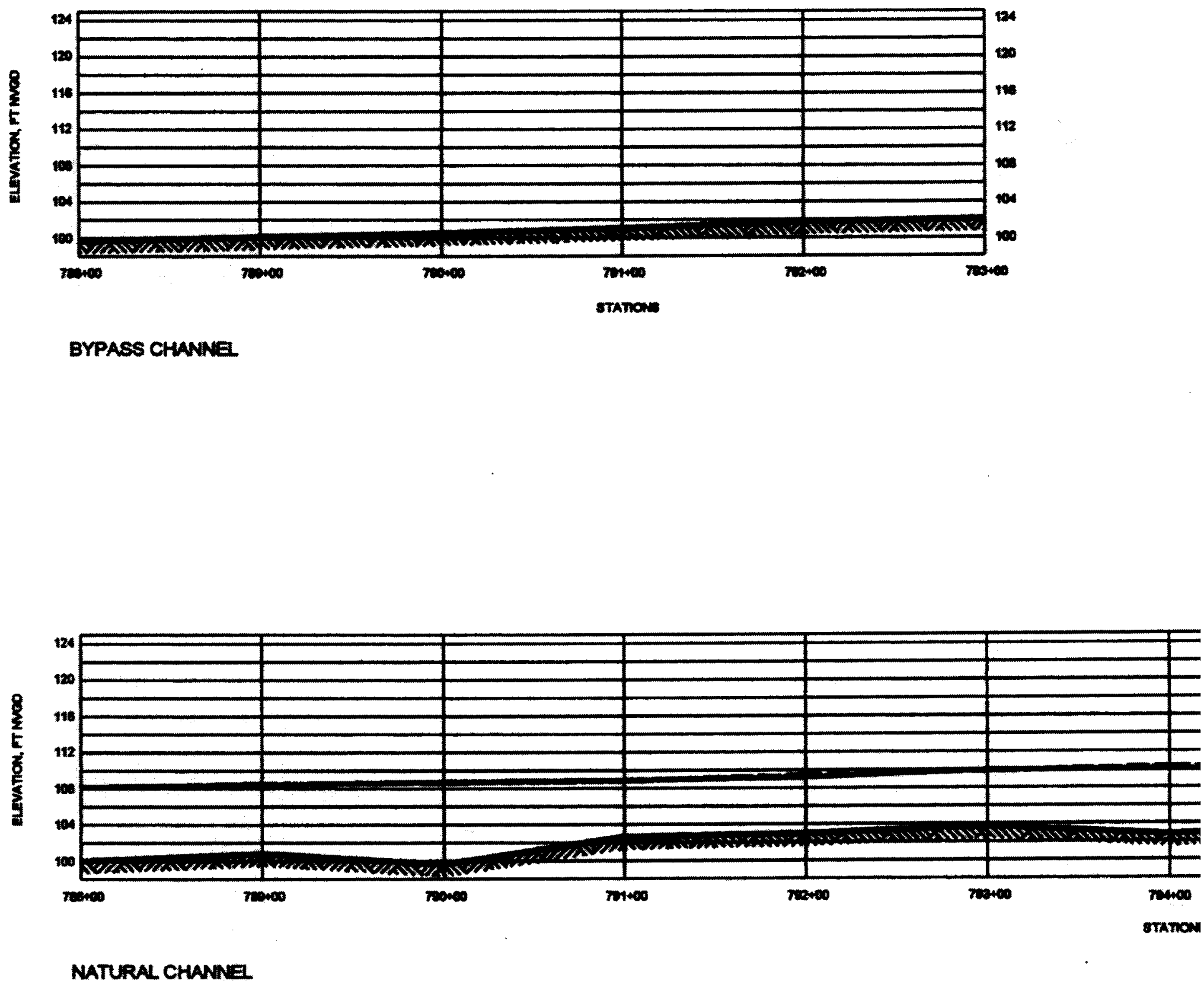
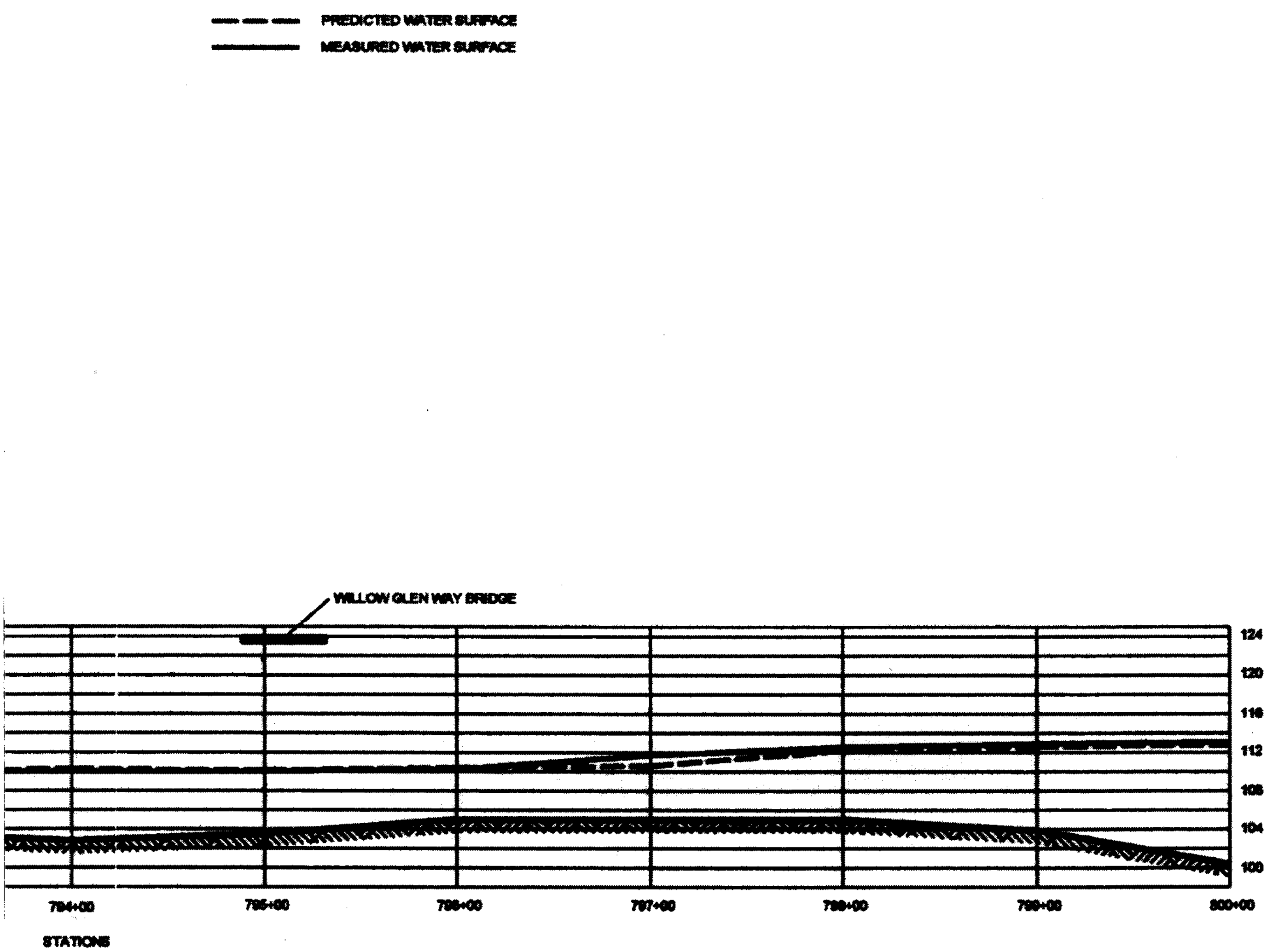
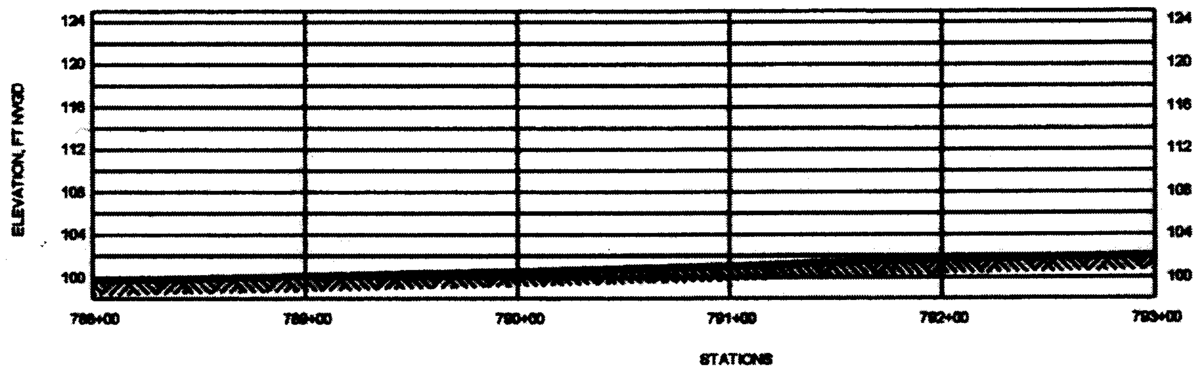

ELEVATION, FT NVGD
124
120
116
112
108
104
100
STATIONS
BYPASS CHANNEL
NATURAL CHANNEL

Figure C2.

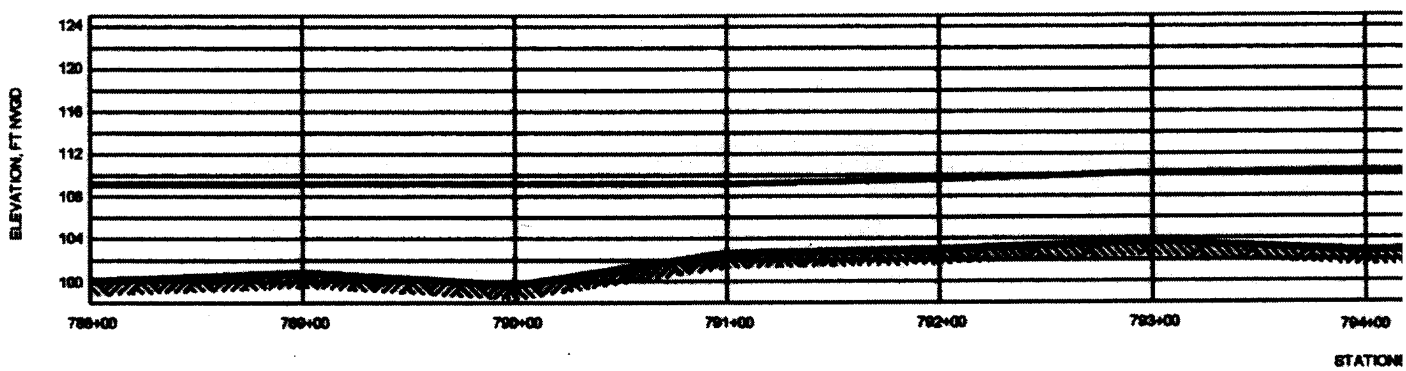


WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER
(BYPASS CHANNEL 0 CFS - NATURAL CHANNEL 1500 CFS)

2



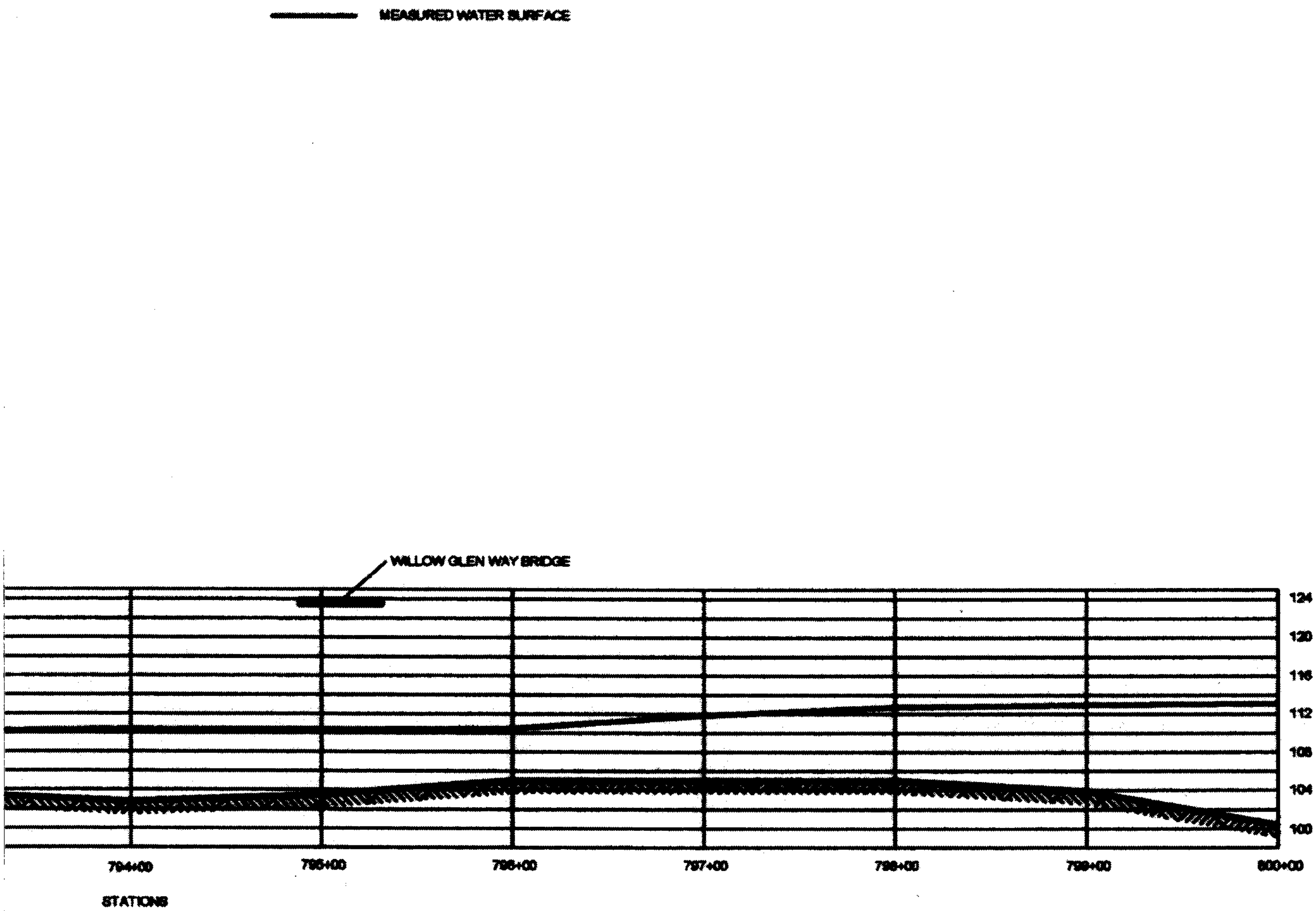
BYPASS CHANNEL



STATION

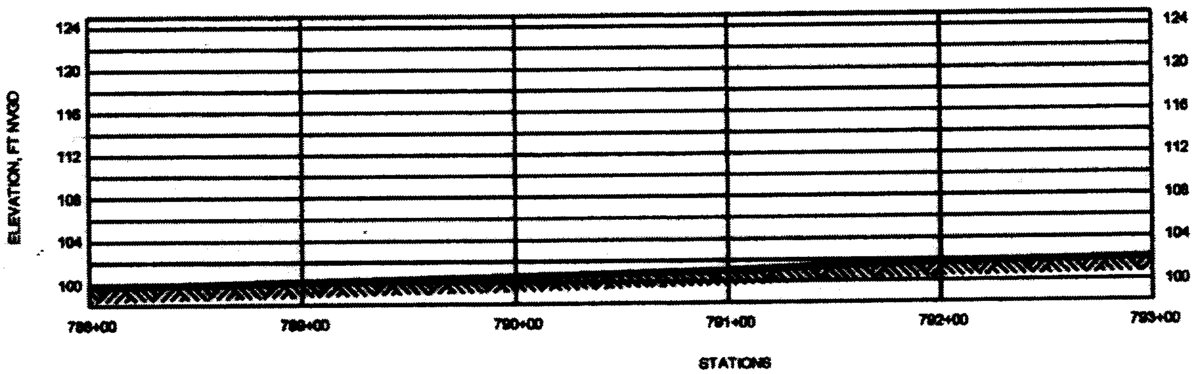
NATURAL CHANNEL

Figure C3.

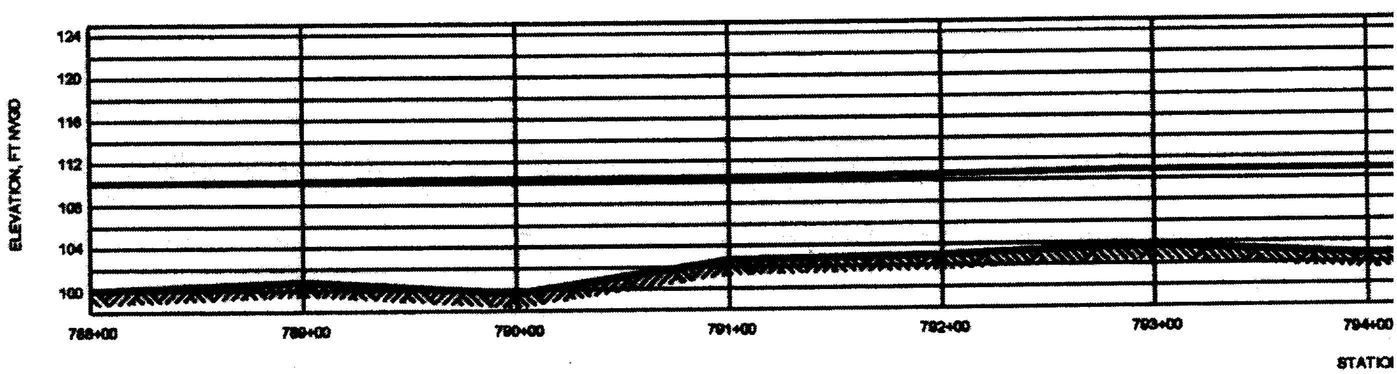


WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 1500 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 0 CFS - NATURAL CHANNEL 1500 CFS)

2

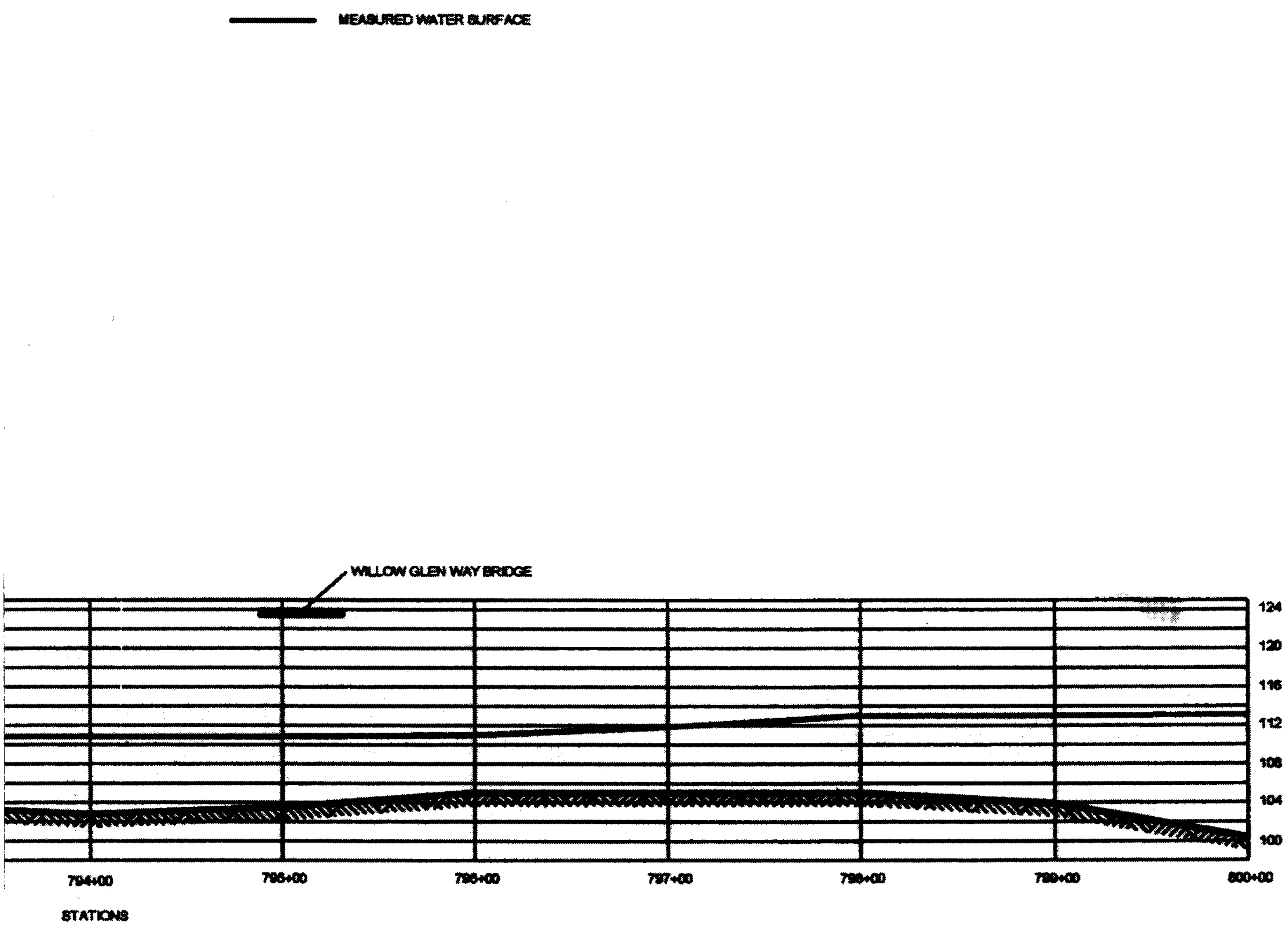


BYPASS CHANNEL



NATURAL CHANNEL

Figure C4.



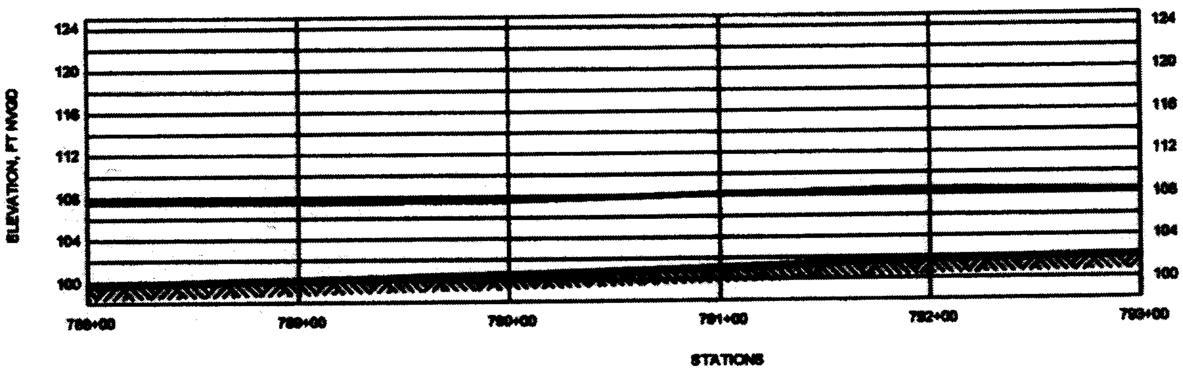
WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN

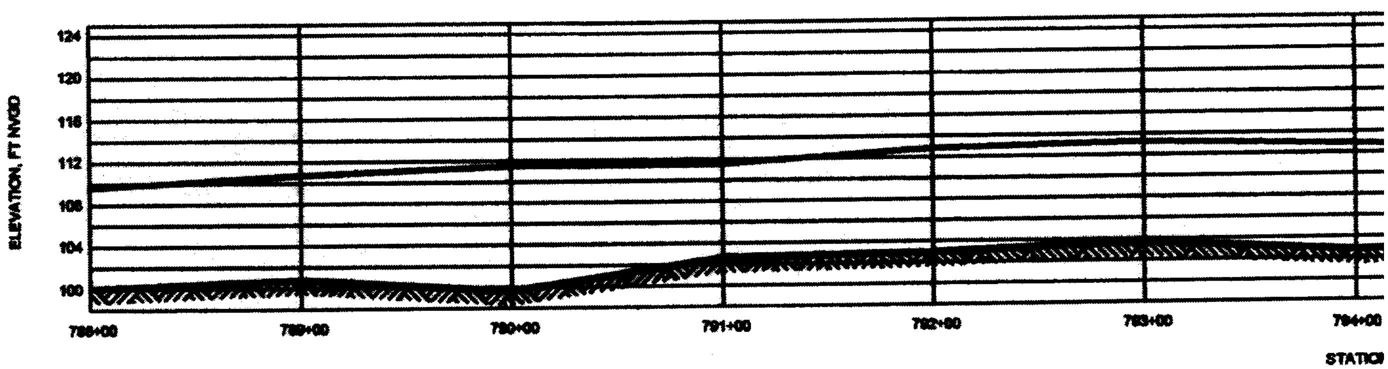
DISCHARGE 1500 CFS DESIGN TAILWATER +2.0'

(BYPASS CHANNEL 100 CFS - NATURAL CHANNEL 1400 CFS)

2

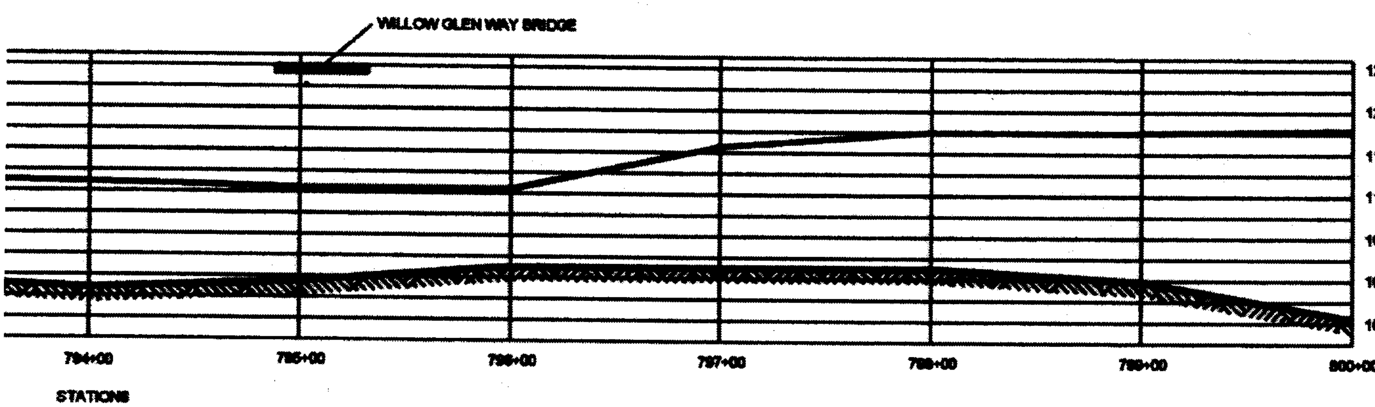


BYPASS CHANNEL



NATURAL CHANNEL

Figure C5.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 7100 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 3500 CFS - NATURAL CHANNEL 3600 CFS)

2

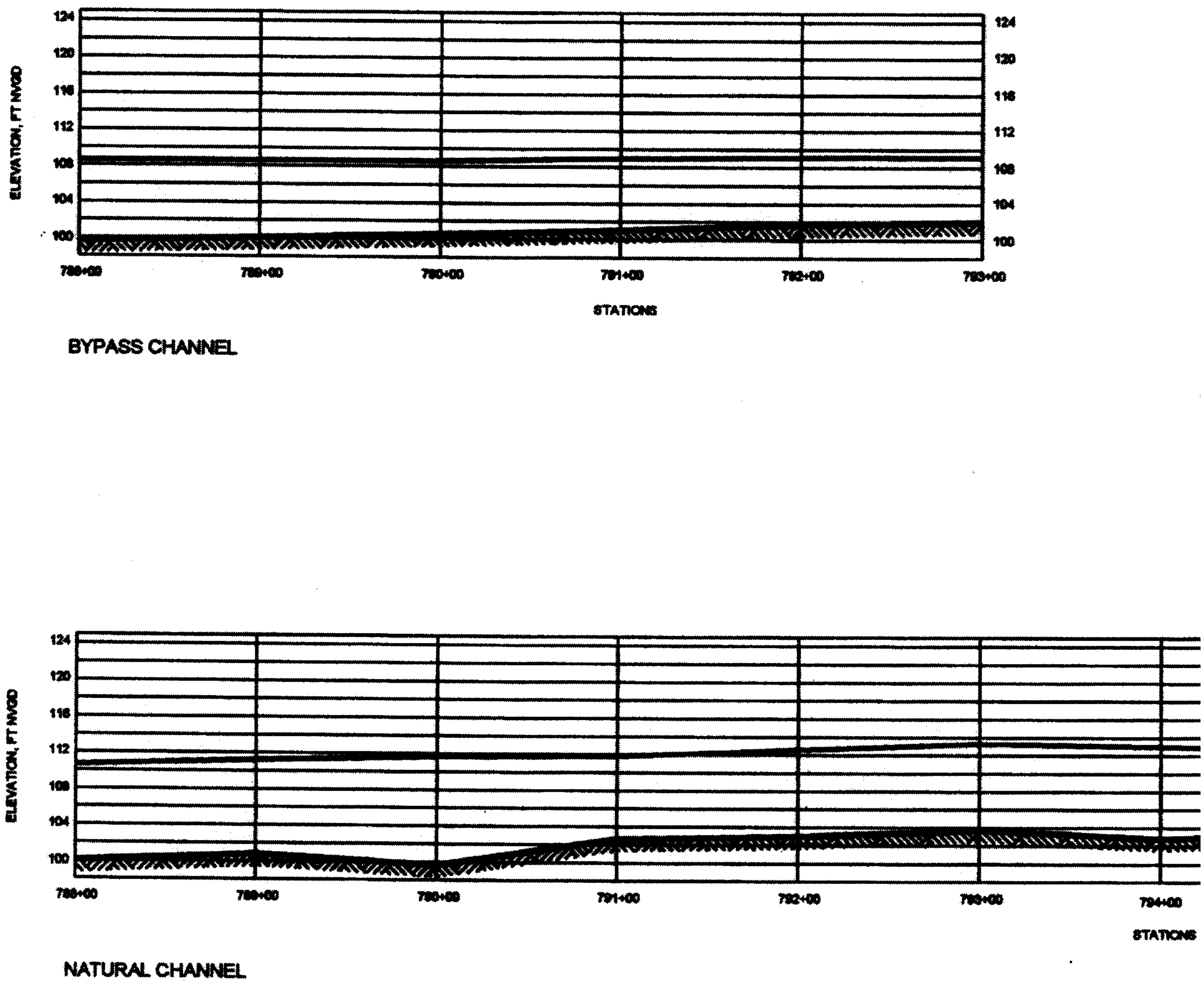
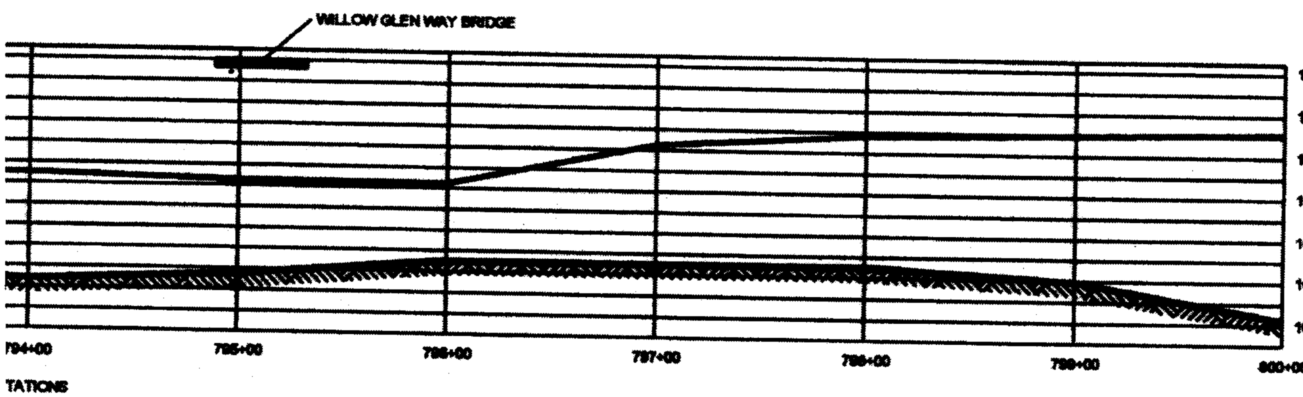

ELEVATION, FT NVGD
124
120
116
112
108
104
100
STATIONS
BYPASS CHANNEL
ELEVATION, FT NVGD
124
120
116
112
108
104
100
STATIONS
NATURAL CHANNEL

Figure C6.



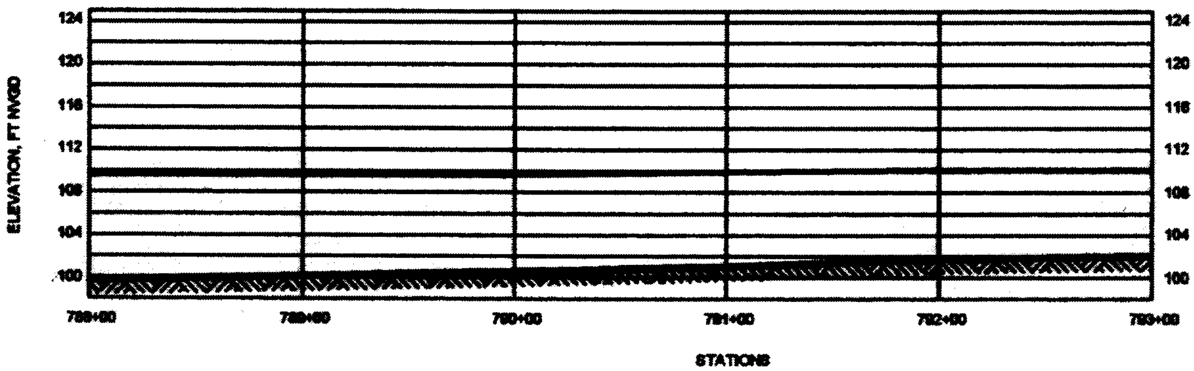
WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN

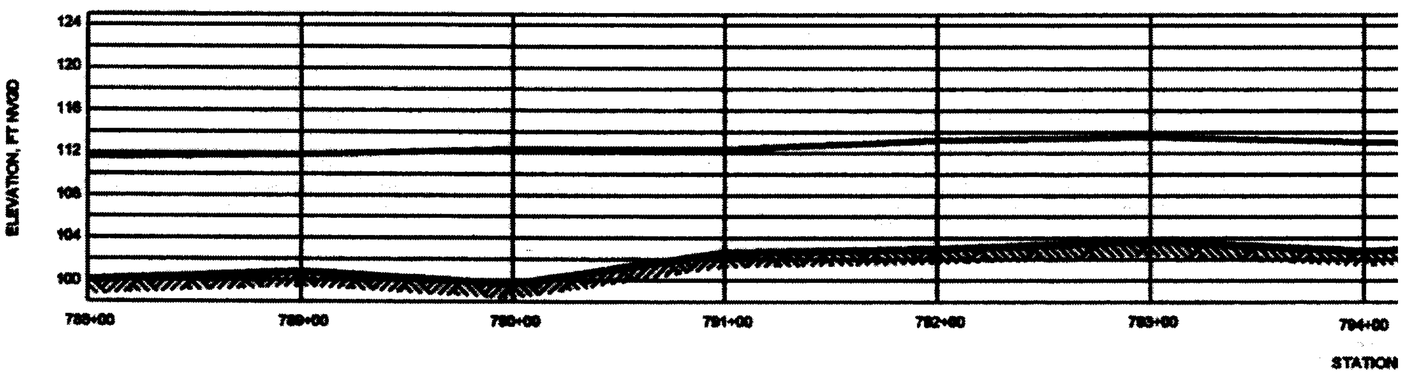
DISCHARGE 7100 CFS DESIGN TAILWATER

(BYPASS CHANNEL 3500 CFS - NATURAL CHANNEL 3600 CFS)

2

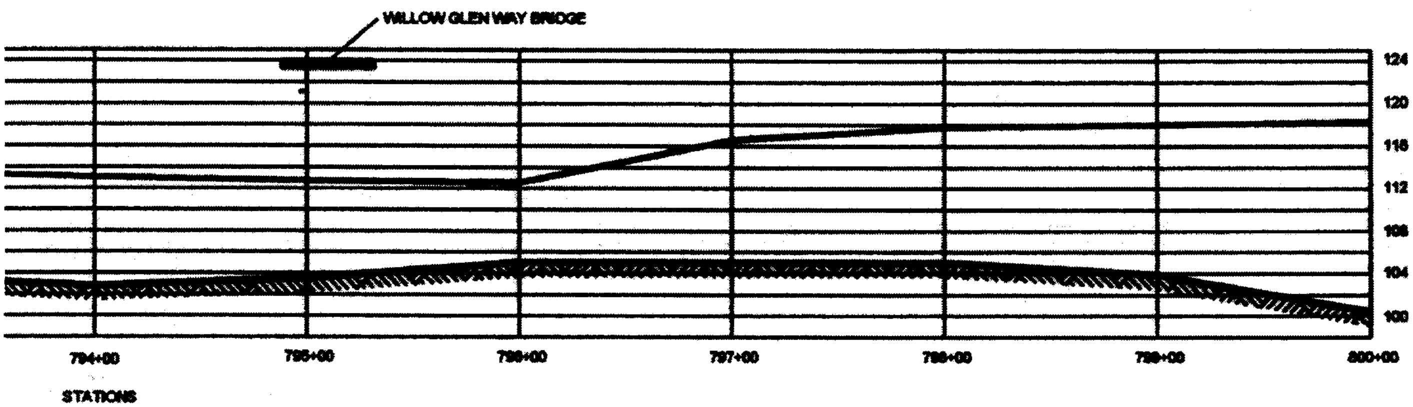


BYPASS CHANNEL



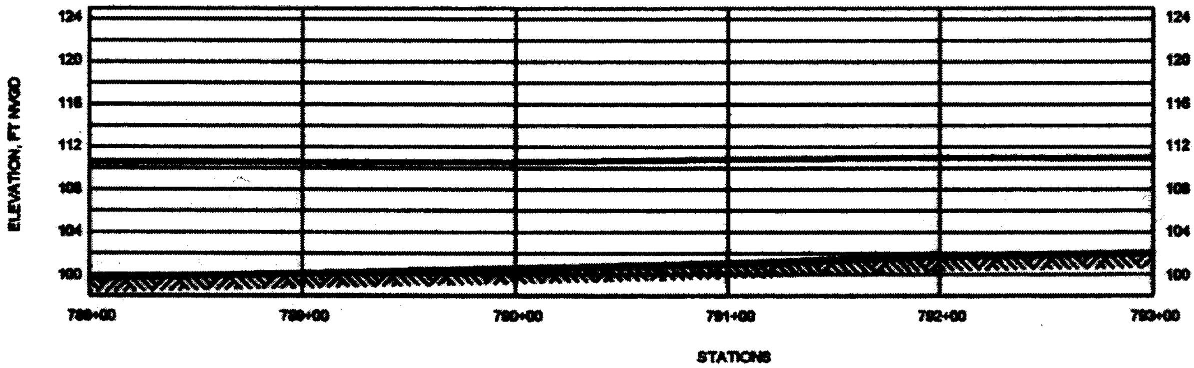
NATURAL CHANNEL

Figure C7.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 7100 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 3650 CFS - NATURAL CHANNEL 3450 CFS)

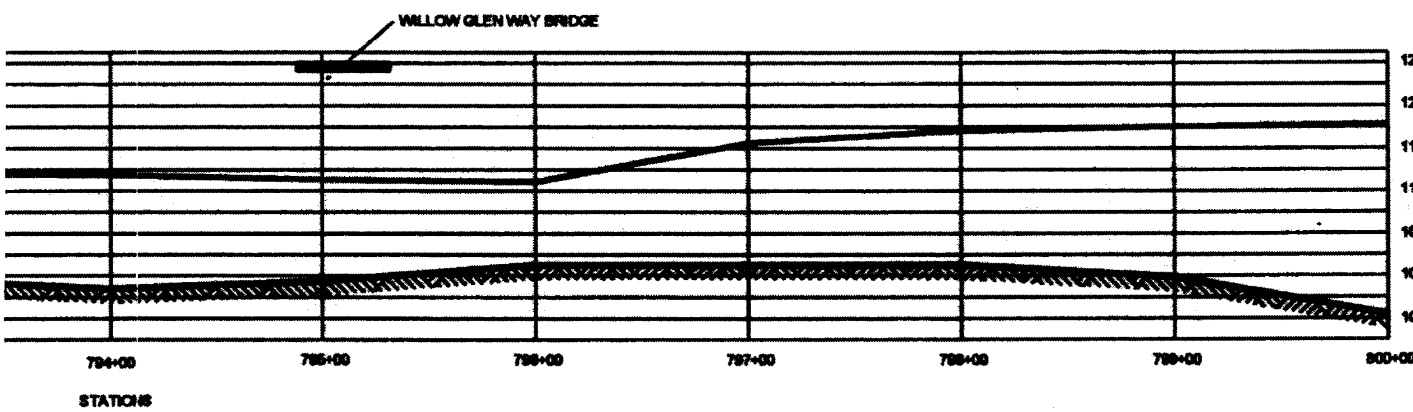
2



BYPASS CHANNEL

NATURAL CHANNEL

Figure C8.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 7100 CFS DESIGN TAILWATER +2.0'
(BYPASS CHANNEL 3800 CFS - NATURAL CHANNEL 3300 CFS)

2

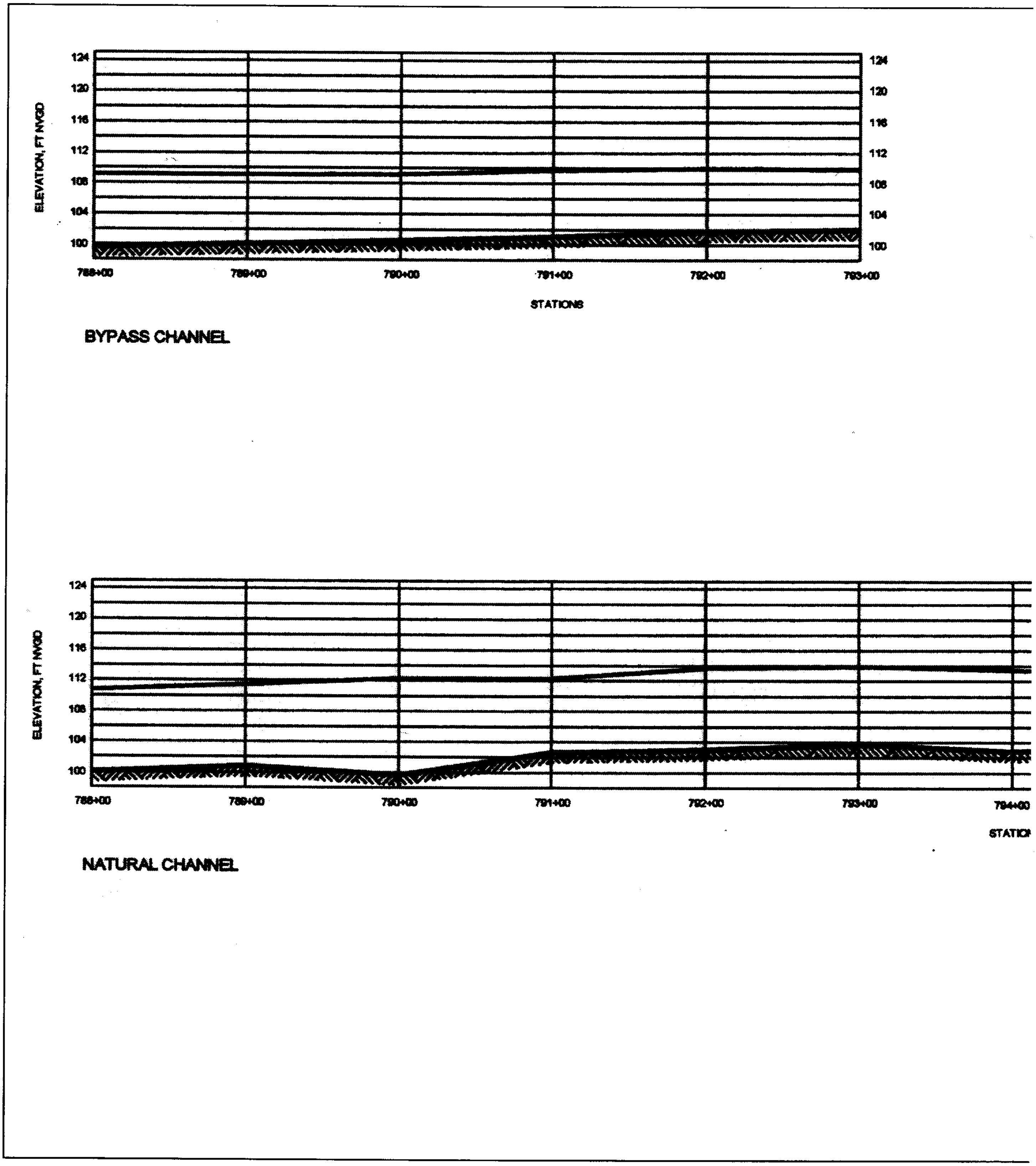
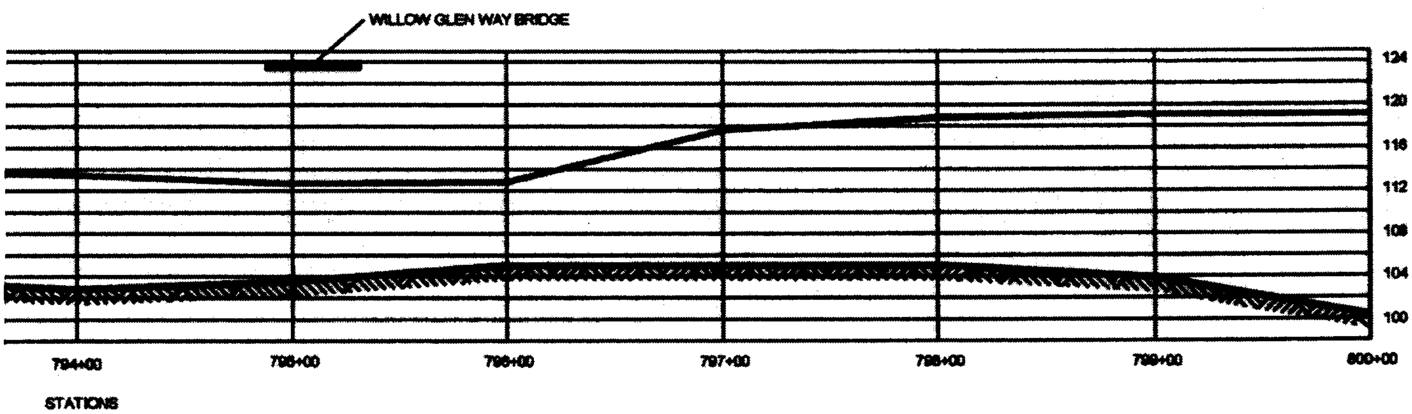

124
120
116
112
108
104
100
ELEVATION, FT NVGD
788+00
789+00
790+00
791+00
792+00
793+00
STATIONS
BYPASS CHANNEL
124
120
116
112
108
104
100
ELEVATION, FT NVGD
788+00
789+00
790+00
791+00
792+00
793+00
794+00
NATURAL CHANNEL

Figure C9.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 9000 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 4850 CFS - NATURAL CHANNEL 4150 CFS)

2

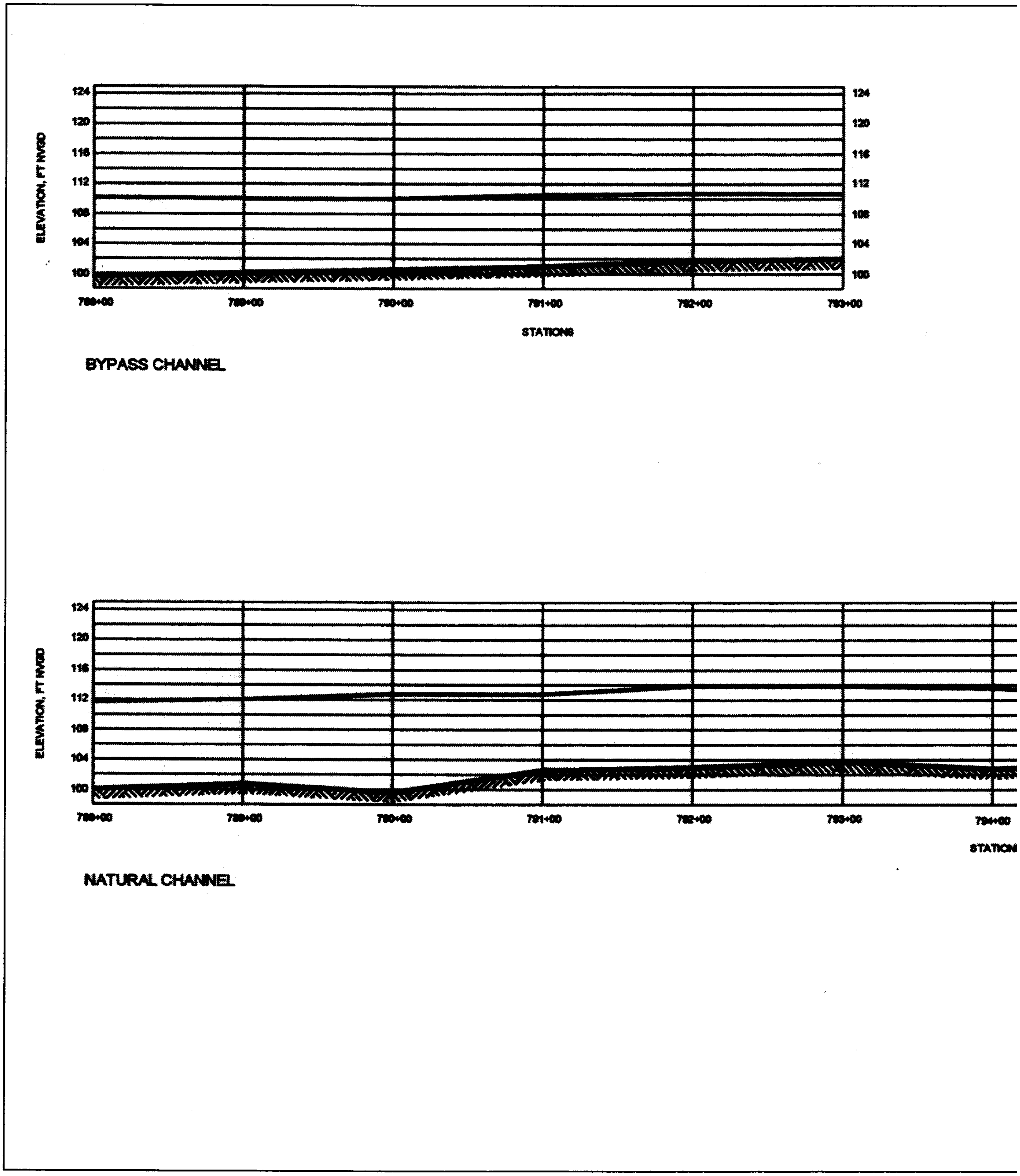
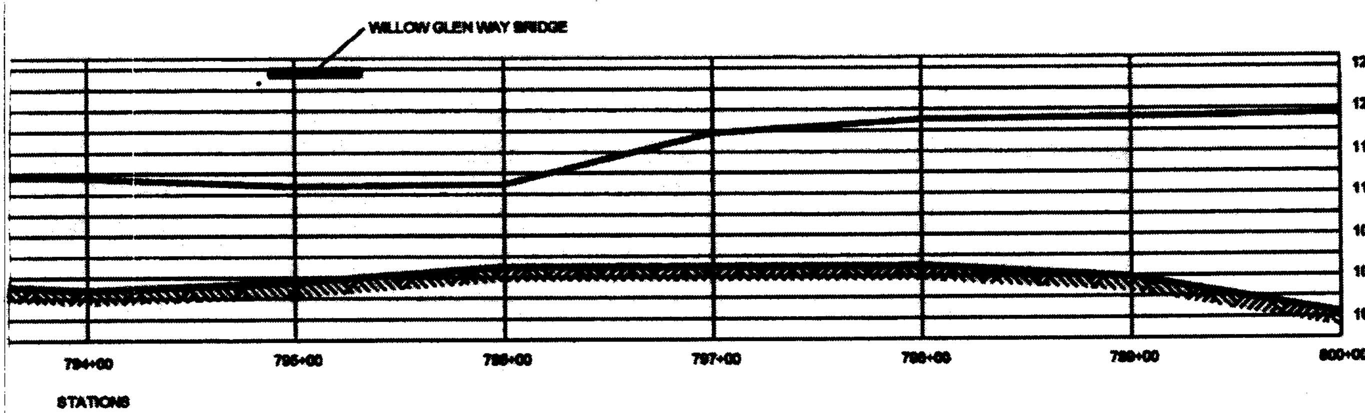

ELEVATION, FT NVGD
124
120
116
112
108
104
100
STATIONS
BYPASS CHANNEL
ELEVATION, FT NVGD
124
120
116
112
108
104
100
NATURAL CHANNEL

Figure C10.



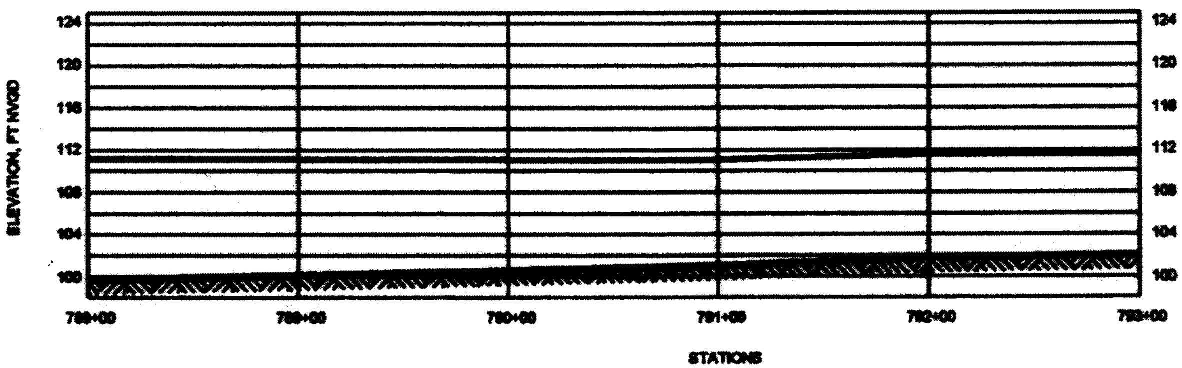
WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN

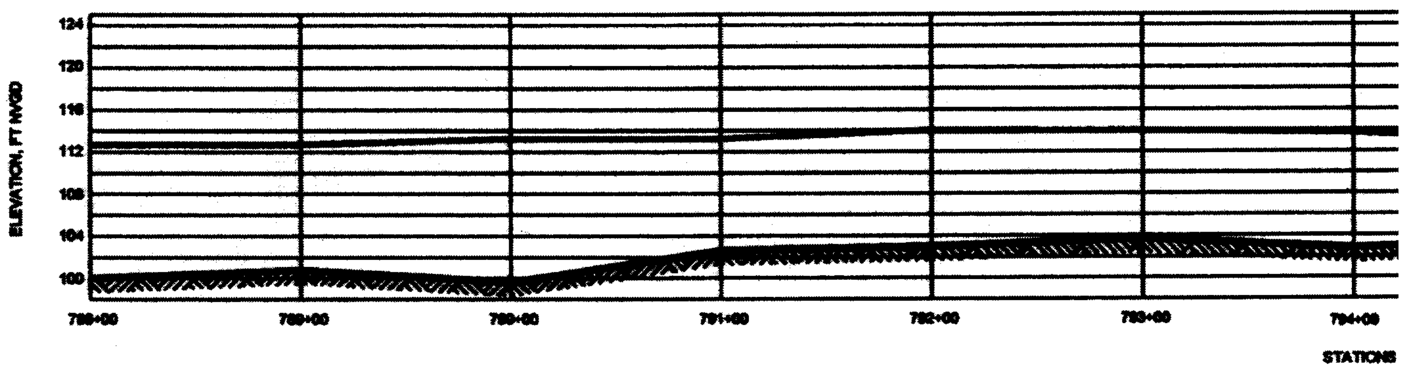
DISCHARGE 9000 CFS DESIGN TAILWATER

(BYPASS CHANNEL 4900 CFS - NATURAL CHANNEL 4100 CFS)

2

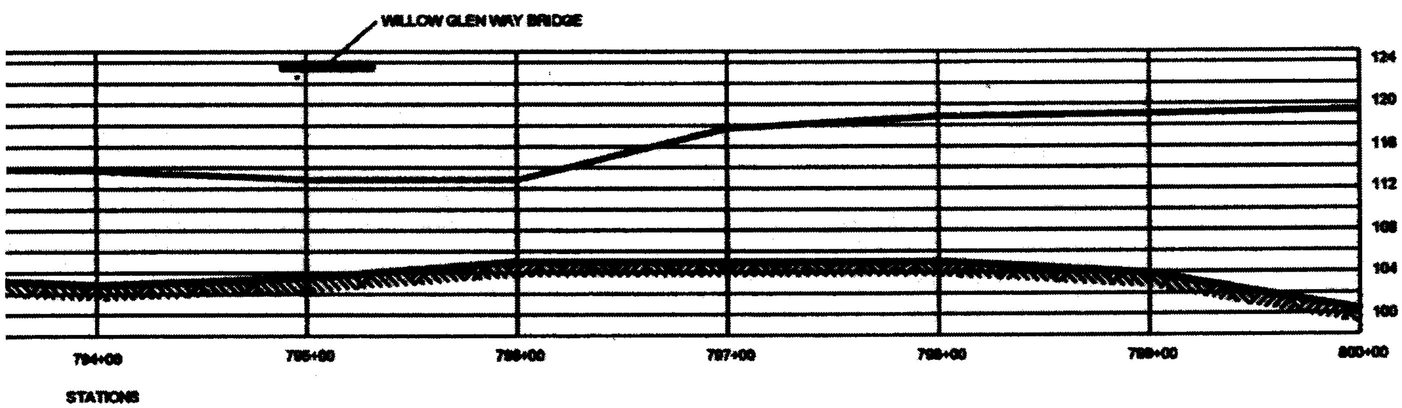


BYPASS CHANNEL



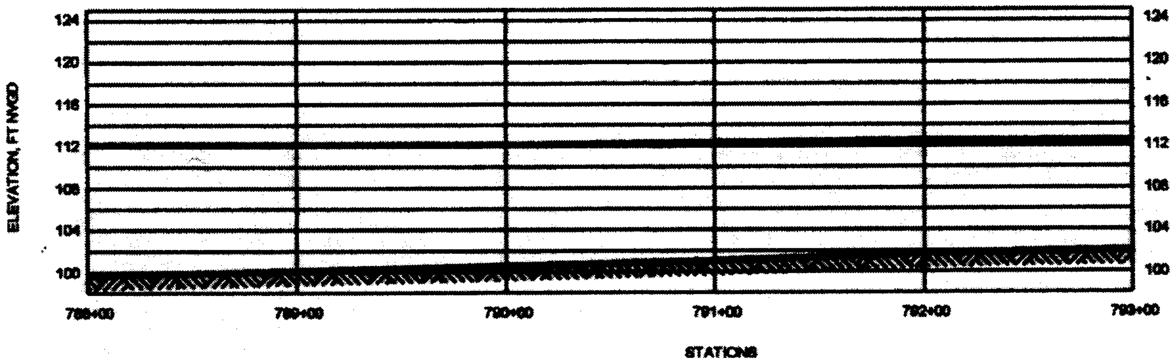
NATURAL CHANNEL

Figure C11.

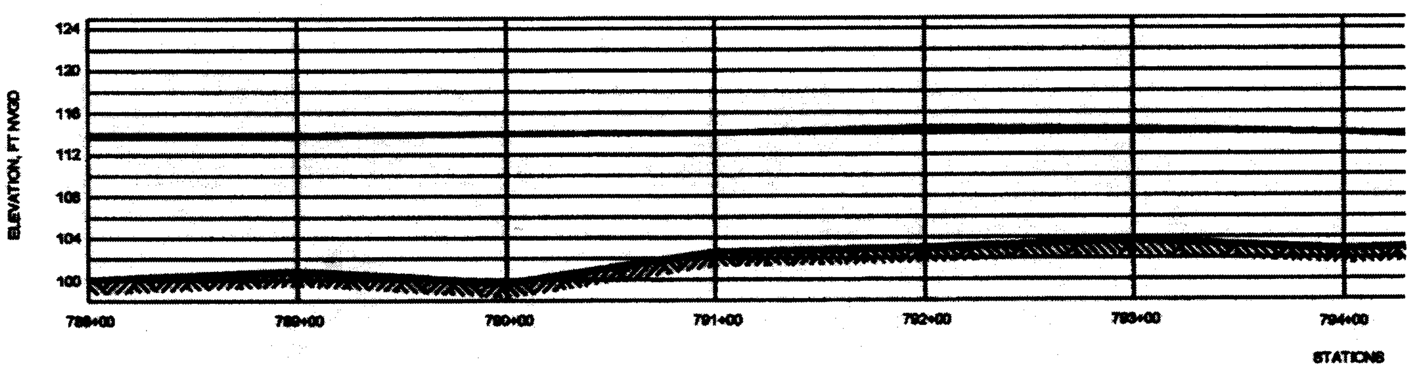


WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 9000 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 5000 CFS - NATURAL CHANNEL 4000 CFS)

2

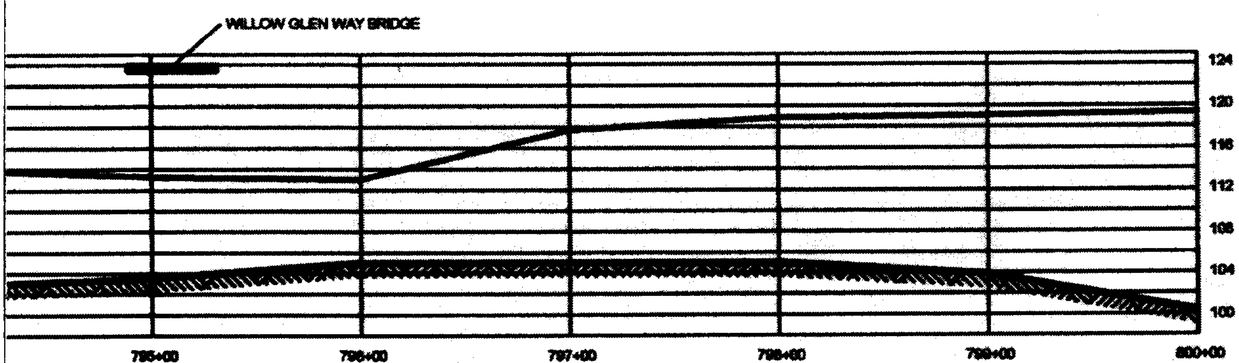


BYPASS CHANNEL



NATURAL CHANNEL

Figure C12.

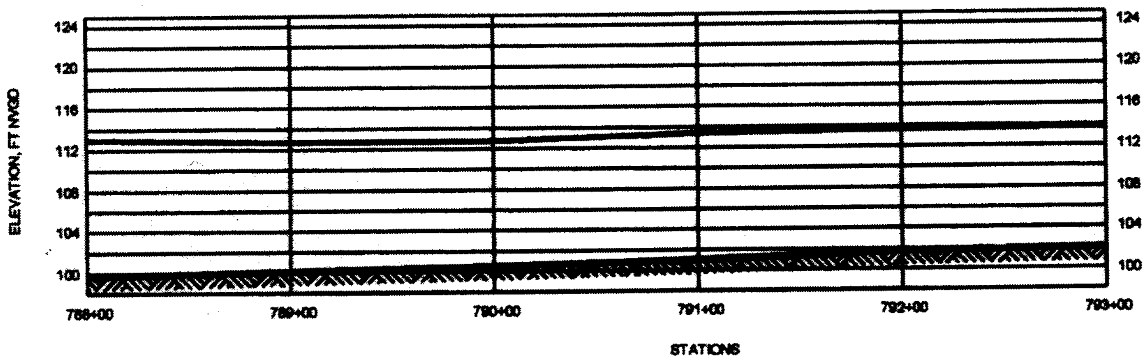


WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

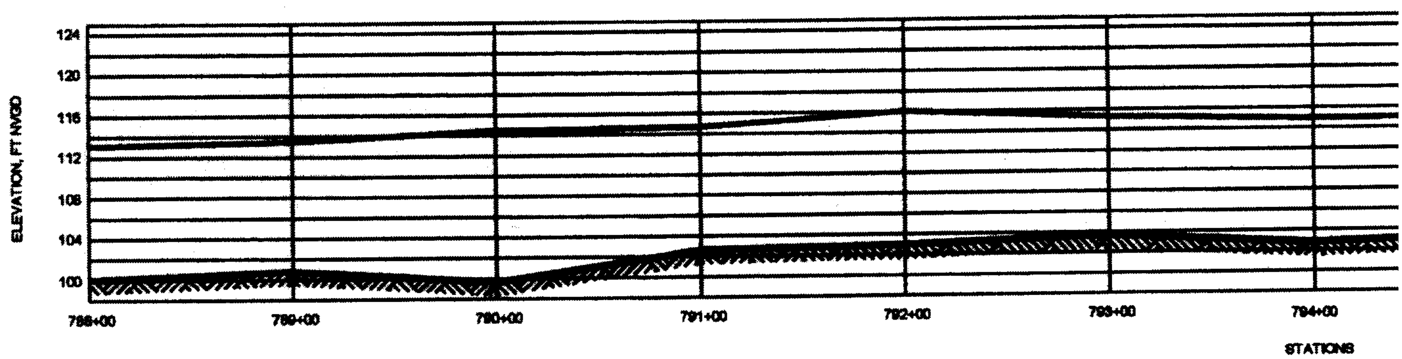
ALTERNATIVE "FINAL" WEIR DESIGN

DISCHARGE 9000 CFS DESIGN TAILWATER +2.0'

(BYPASS CHANNEL 5350 CFS - NATURAL CHANNEL 3650 CFS)

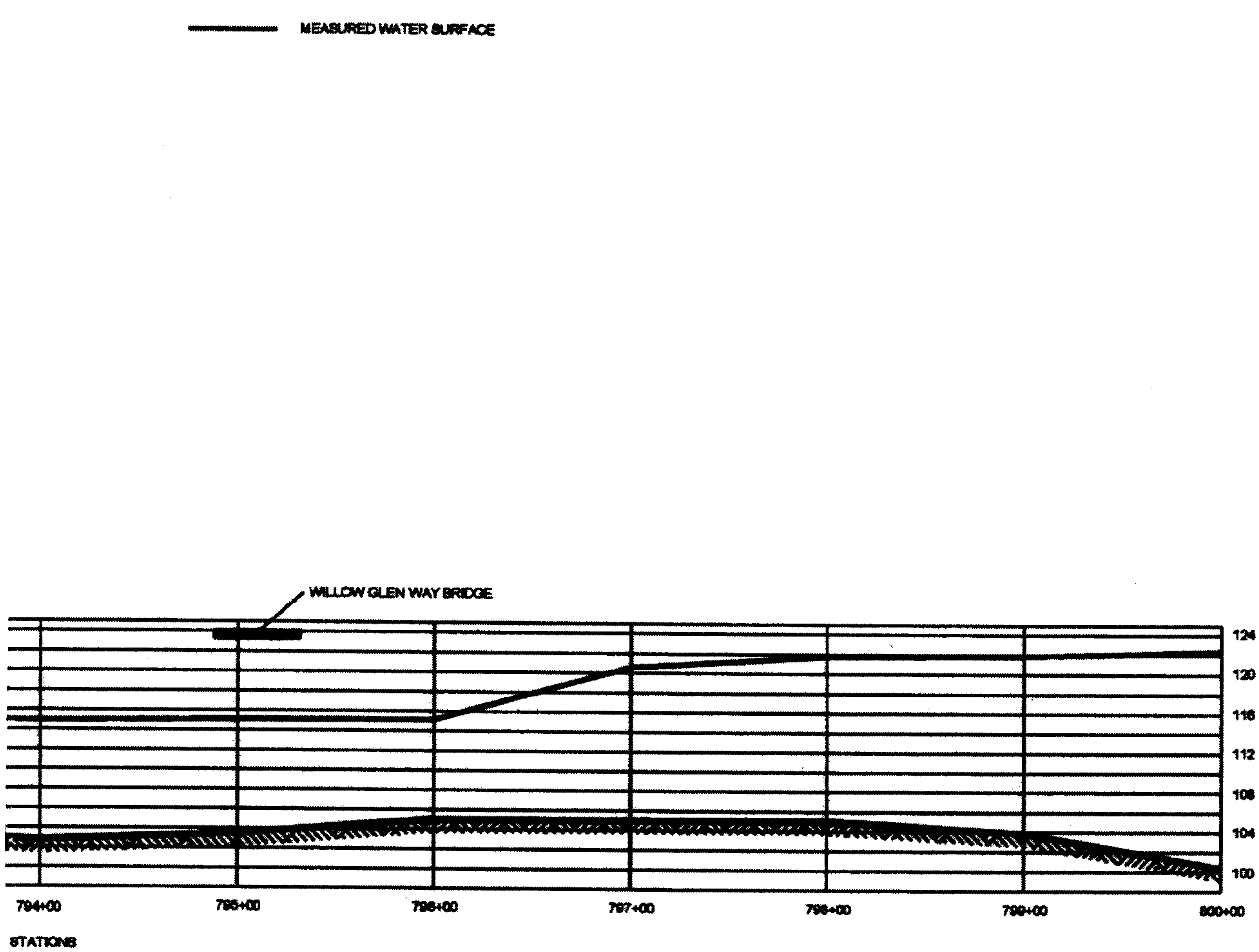


BYPASS CHANNEL



NATURAL CHANNEL

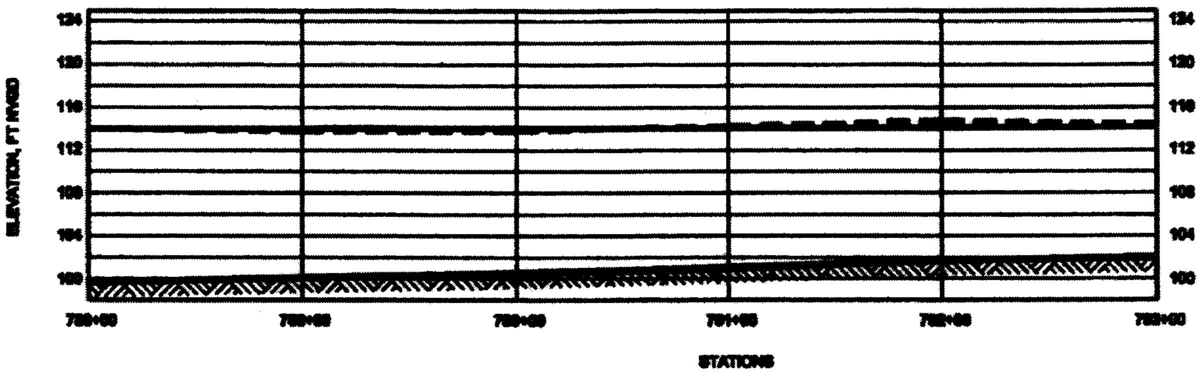
Figure C13.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS

ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 14600 CFS DESIGN TAILWATER -1.0'
(BYPASS CHANNEL 8900 CFS - NATURAL CHANNEL 5700 CFS)

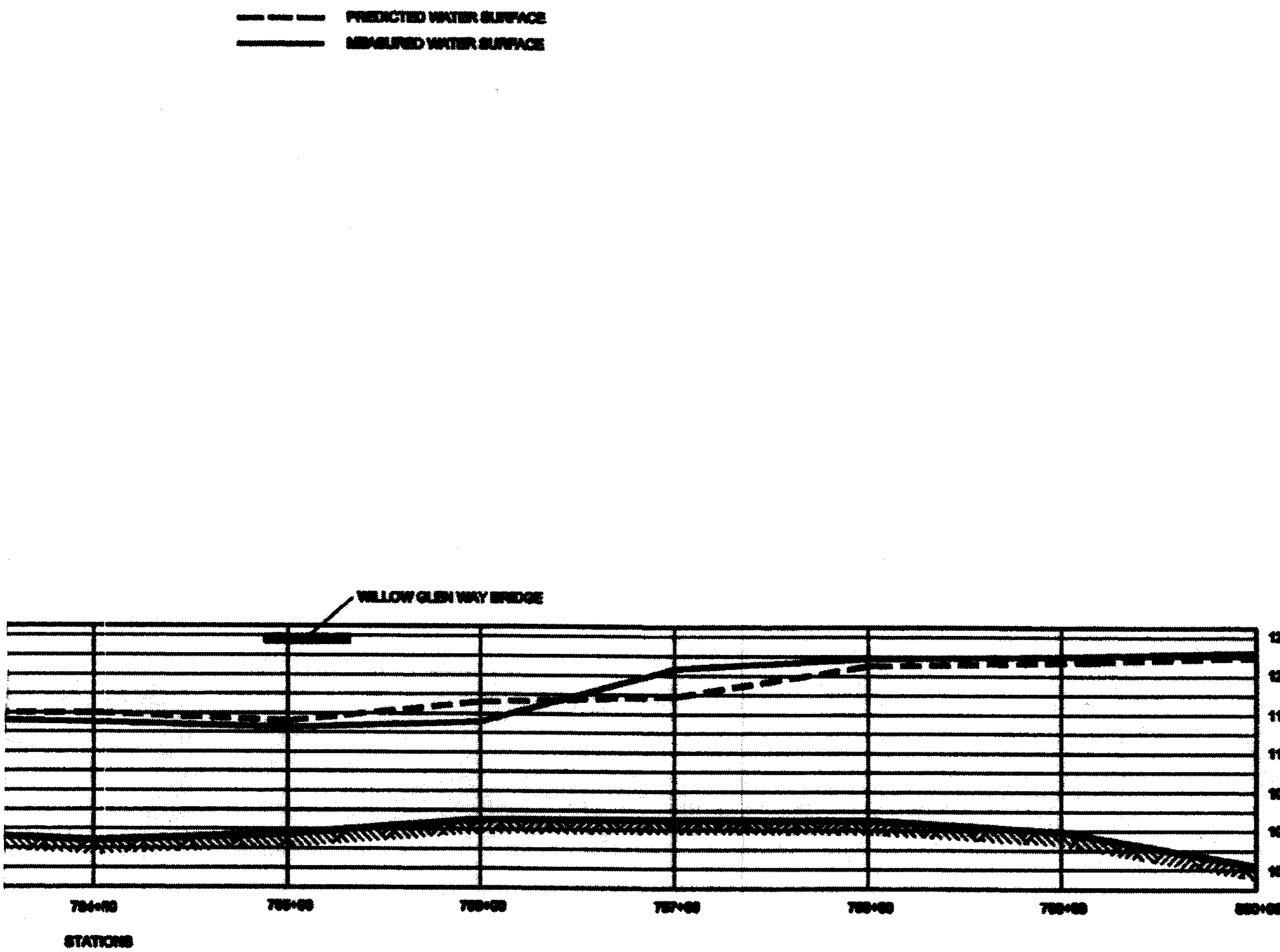
2



BYPASS CHANNEL

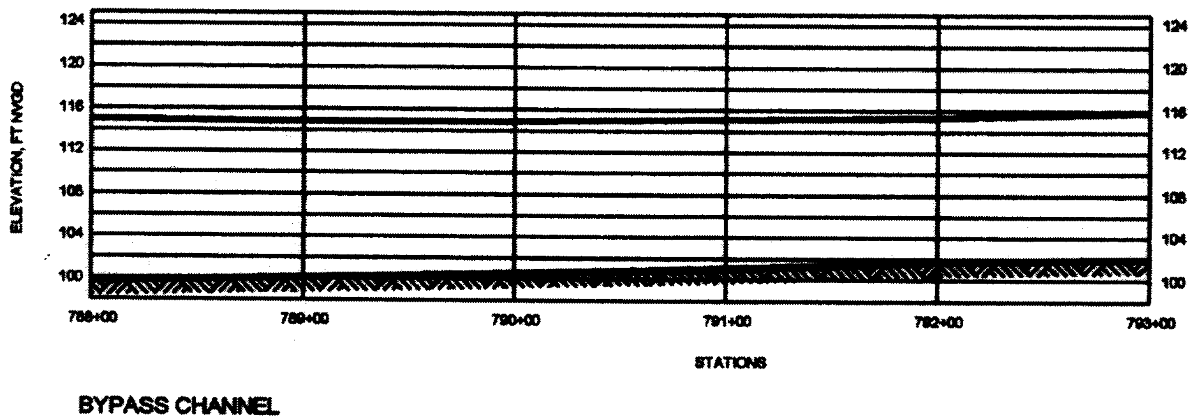
NATURAL CHANNEL

Figure C14.

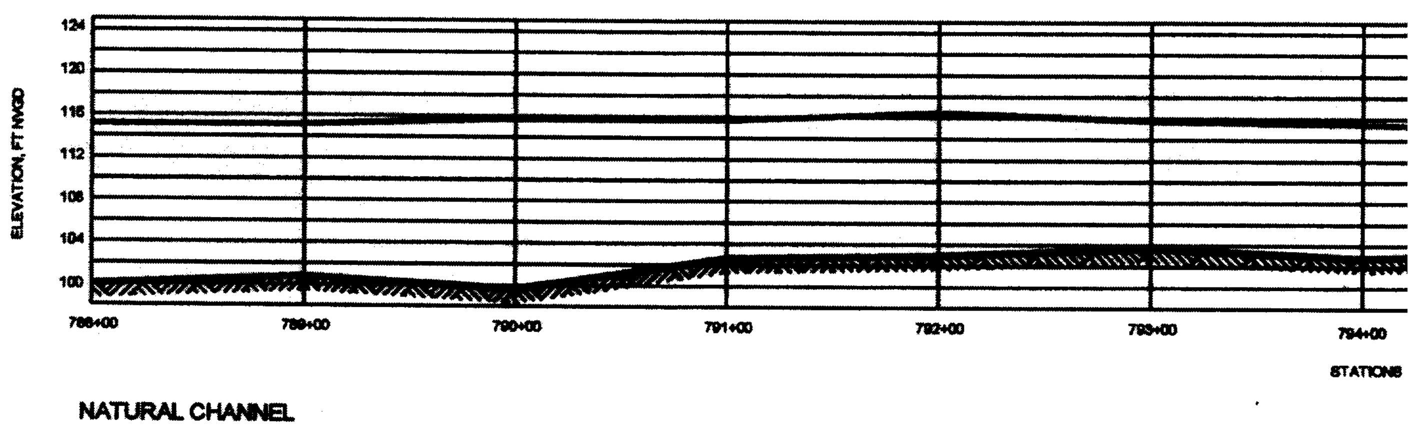


WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 14600 CFS DESIGN TAILWATER
(BYPASS CHANNEL 8900 CFS - NATURAL CHANNEL 5700 CFS)

2

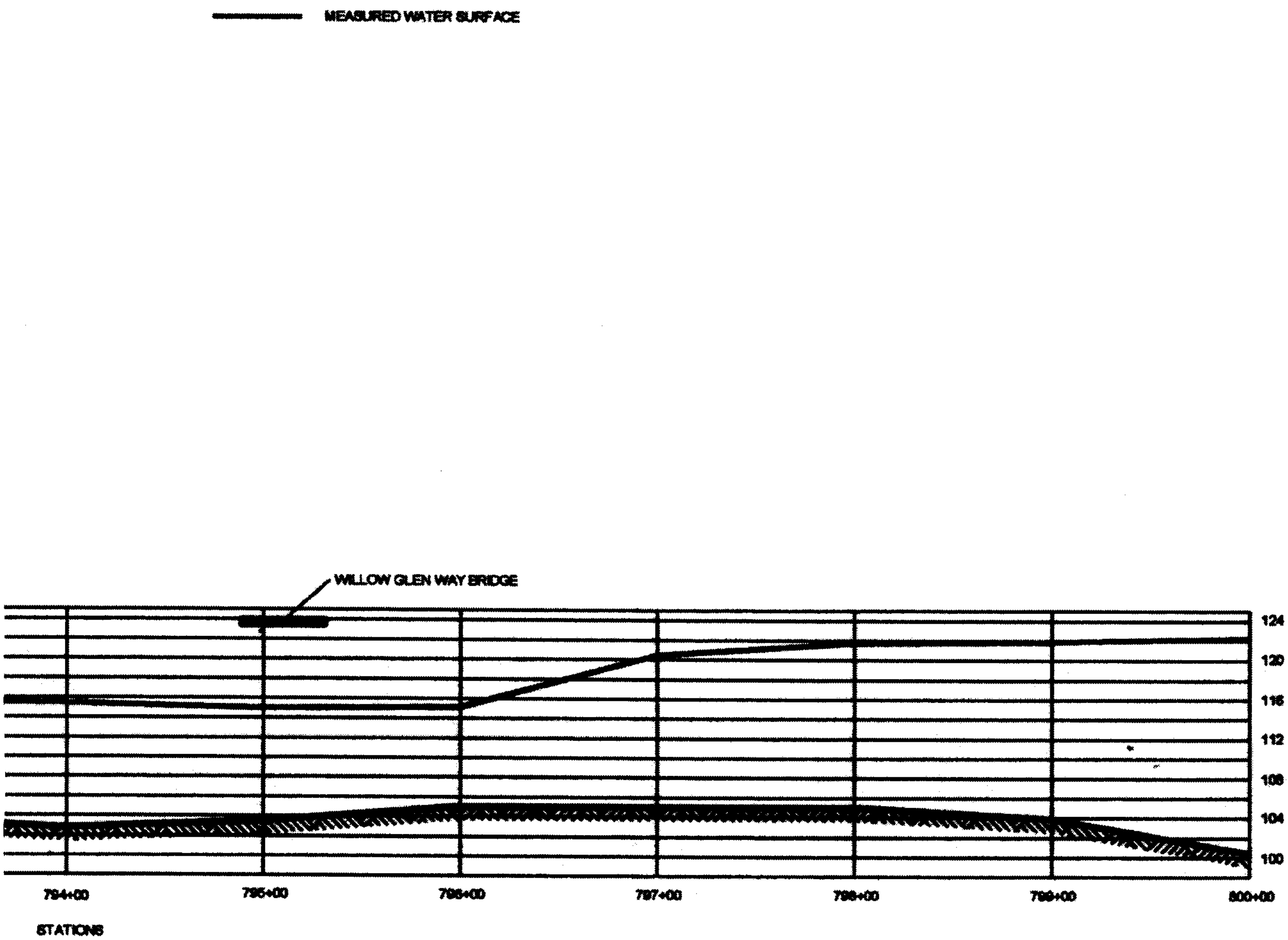


ELEVATION, FT NVGD
124
120
116
112
108
104
100
788+00
789+00
790+00
791+00
792+00
793+00
STATIONS
BYPASS CHANNEL



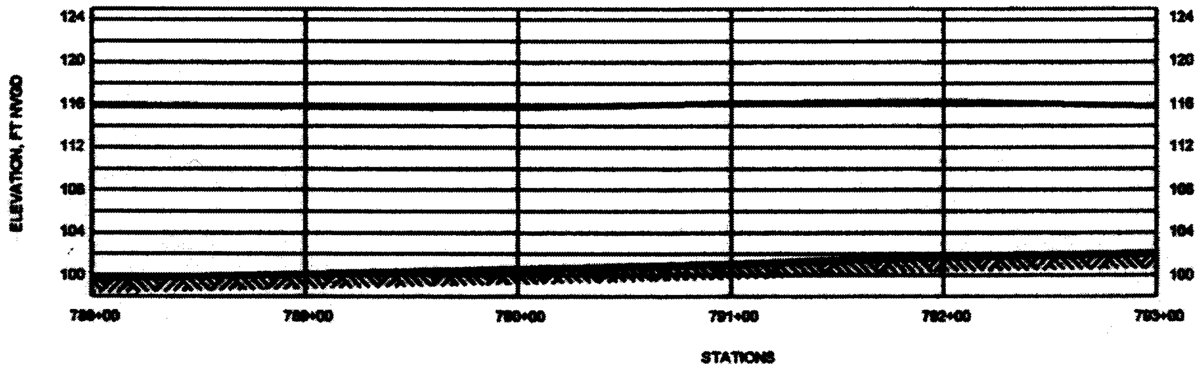
ELEVATION, FT NVGD
124
120
116
112
108
104
100
788+00
789+00
790+00
791+00
792+00
793+00
794+00
STATIONS
NATURAL CHANNEL

Figure C15.

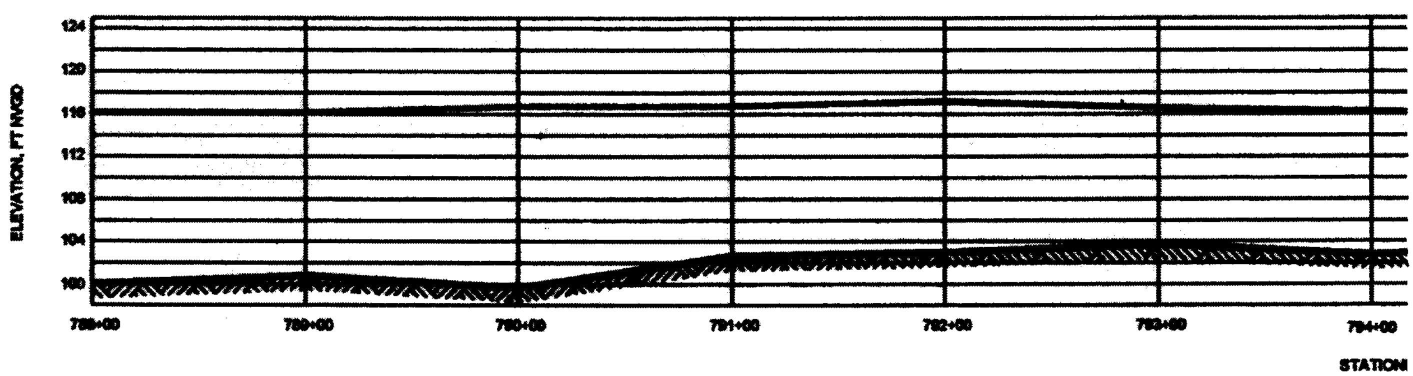


WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 14600 CFS DESIGN TAILWATER +1.0'
(BYPASS CHANNEL 8800 CFS - NATURAL CHANNEL 5800 CFS)

2

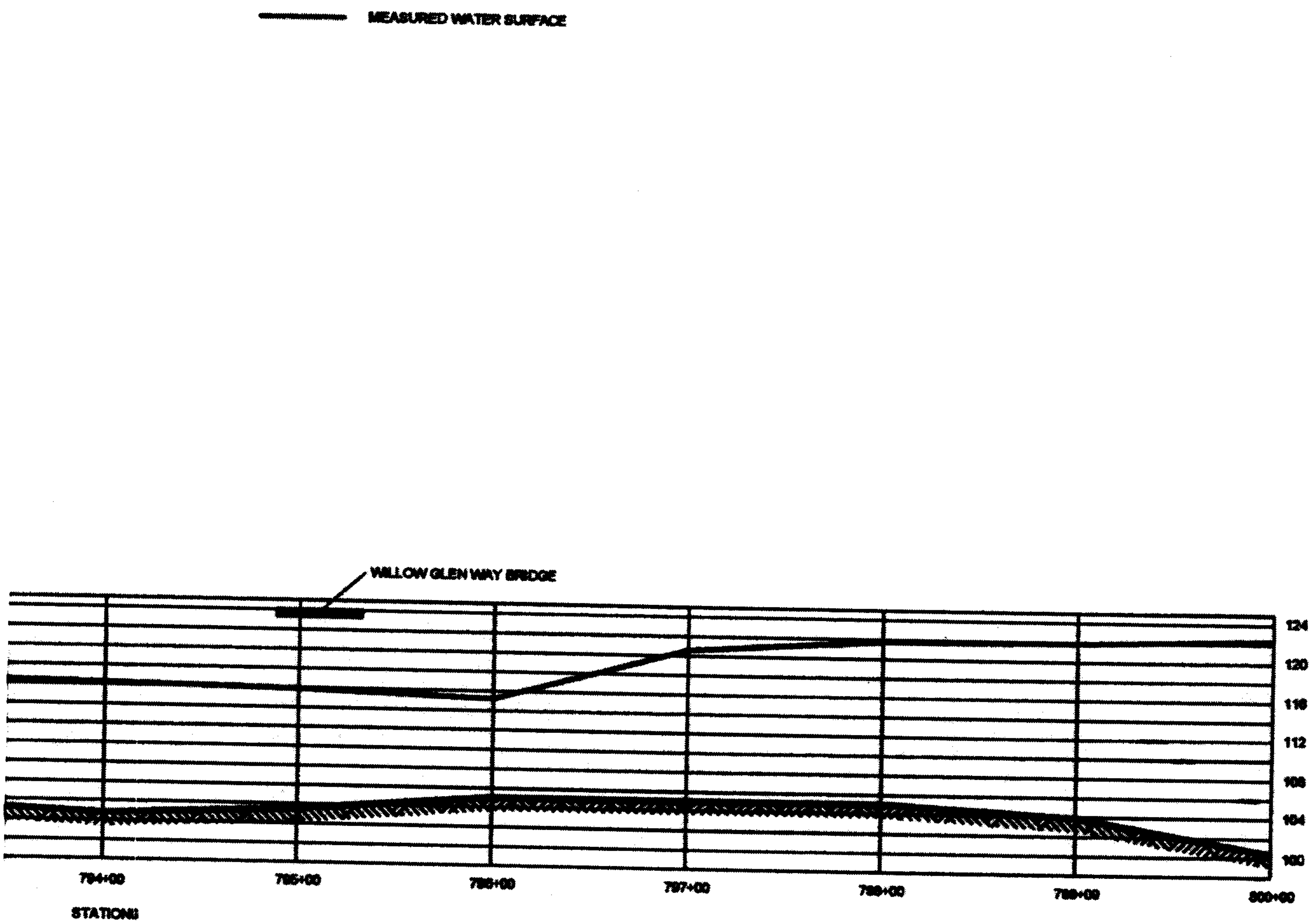


BYPASS CHANNEL



NATURAL CHANNEL

Figure C16.



WATER-SURFACE PROFILES
NEW CHANNEL ROUGHNESS
ALTERNATIVE "FINAL" WEIR DESIGN
DISCHARGE 14600 CFS DESIGN TAILWATER +2.0'
(BYPASS CHANNEL 8600 CFS - NATURAL CHANNEL 6000 CFS)

2

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November 2003

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6. AUTHOR(S)
Billy D. Fuller

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14. ABSTRACT

Tests were conducted on a 1:36 scale model of a portion of the Guadalupe River. The study was designed to investigate the design of a control weir located at the upstream end of a bypass channel. The bypass channel was designed to pass excess flow and prevent flooding for river flows up to a 100-year event. The original weir design was modified to achieve desired flow distribution and control.

15. SUBJECT TERMS

Bridge pier	Channel roughness	Design roughness	Sediment	Weir
Bypass channel	Channel velocities	Flood flow	Tailwater sensitivity	Weir coefficient
	Control structure	Flow distribution	Water-surface profiles	

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